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DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

PROCEEDINGS
of the
NINTH ANNUAL MEETING
of the
HIGHWAY RESEARCH BOARD

Held at Washington, D. C.
December 12-13, 1929

Edited by
ROY W. CRUM
Director, Highway Research Board

WASHINGTON, D. C.
1930

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DEVELOPMENT OF HIGHWAY RESEARCH BOARD

The National Research Council is a cooperative organization of scientific and technical men of America, including also business men interested in science and industry. In 1916, at a time when it seemed probable that America might enter the war, the National Academy of Sciences offered its services to the President of the United States, who accepted the offer, and requested the Academy to organize the scientific resources of the country. The Academy met the request and, under its Congressional Charter, organized the National Research Council. On May 11, 1918, President Wilson requested the Academy to perpetuate the Council as a permanent organization. The Council, reorganized at that time, and adapted to peace-time conditions, has continued to coordinate and promote scientific research for the benefit of the nation as a whole.

The Advisory Board on Highway Research was organized by the Council through its Division of Engineering on November 11, 1920. The name of the organization was later changed to Highway Research Board.

The functions of this Highway Research Board are: To prepare a comprehensive national program for highway research; to assist existing organizations to coordinate their activities therein; and to serve as a clearing house for information on completed and current research.

ORGANIZATION AND MEMBERSHIP

The Highway Research Board is composed of representatives of national organizations interested in the development of highways of the country. Connection with all phases of highway industry is maintained by means of research committees, and contact men appointed from the various state highway departments, universities and colleges.

The Highway Research Board functions through an executive committee and a director, who is the executive officer.

The Highway Research Board also conducts special investigations financed by interested organizations, and furnishes unique auspices under which work may be carried on that is scientific in character and impartial in its findings, and which is generally accepted as authentic.

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Chas. M. Upham, Secretary-Director, American Road Builders' Association.

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    J. L. Janda

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    R. D. Goodrich

NINTH ANNUAL MEETING OF THE HIGHWAY RESEARCH BOARD

PROGRAM

Thursday, December 12

MORNING SESSION, 10:00 A. M.

F. H. Eno, *Chairman,* Presiding

Address of Welcome..Albert L. Barrows
Assistant Secretary, National Research Council

Response..F. H. Eno
Chairman, Highway Research Board

A Proposed Program of Highway Research..................T. H. MacDonald
Chief, United States Bureau of Public Roads

Report of Director and Report of Committee on Coordination and Program ....R. W. Crum

"Application of Aerial Mapping to Highway Construction"......S. D. Sarason
Professor, Civil Engineering, Syracuse University

"Soil Investigations in Russia"..............................D. P. Krynine
Professor of Highway Engineering, Moscow Superior Technical School and Moscow Institute of Transportation Engineering

AFTERNOON SESSION 2:00 P. M.

H. S. Mattimore, Presiding

Report of Committee on Highway Traffic Analysis....G. E. Hamlin, *Chairman*
Superintendent of Maintenance, Connecticut Highway Department

"Seasonal Distribution of Traffic"...........................A. N. Johnson
Dean of Engineering, University of Maryland

Discussion...W. H. Connell
Executive Director, Regional Planning Federation of the Philadelphia Tri-State District

"The Mathematical Theory of External Loads on Closed Conduits in the Light of the Latest Experiments"................Anson Marston
President, American Society of Civil Engineers; Dean of Engineering, Iowa State College

Report of Committee on Maintenance..................B. C. Tiney, *Chairman*
Maintenance Engineer, Michigan Highway Department

"Formation of Washboards in Gravel Highways"............Homer J. Dana
Assistant Director, Engineering Station,
State College of Washington

Highway Research Dinner, 7:00 P. M., Thursday
The Washington Hotel
F. H. Eno, *Chairman*, Presiding

"The Pan American Highway Conference of 1929 and Road Conditions in the Latin American Countries"........................Pyke Johnson
National Automobile Chamber of Commerce

Friday, December 13

MORNING SESSION 10:00 A. M.

George E. Hamlin, Presiding

Report of Committee on Structural Design of Roads..A. T. Goldbeck, *Chairman*
Director, Bureau of Engineering, National Crushed Stone Association

"Influence of Climate on Building, Maintenance, and Use of Roads in the United States"..F. H. Eno
Research Professor of Highway Engineering, Ohio State University

Important Developments with Respect to Subgrade Investigations............................C. A. Hogentogler, Henry Aaron
U. S. Bureau of Public Roads

"Control of Materials and Mixtures for Concrete for Pavements"..R. W. Crum
Director, Highway Research Board

Report of Committee on Materials................H. S. Mattimore, *Chairman*
Engineer of Materials, Pennsylvania Highway Department

"Water Control at Concrete Mixers"...........................Bert Myers
Engineer of Materials and Tests, Iowa Highway Commission
and F. C. Lang
Engineer of Materials and Inspection, Minnesota Highway Department

BUSINESS MEETING 12:30 P. M.

(Executive Committee and Representatives of Member Organizations)

F. H. Eno, *Chairman*, Presiding

AFTERNOON SESSION 2:00 P. M.

A. T. Goldbeck, Presiding

Report of Committee on Highway Finance.............H. J. Kirk, *Chairman*
Consulting Engineer

Report of Committee on Highway Transportation Costs..T. R. Agg, *Chairman*
Professor of Highway Engineering, Iowa State College

Progress Report of Special Investigation of "Curing of Concrete Pavement Slabs"..........F. C. Lang, *Chairman*
Fred Burggraf, *Special Investigato*

Report of Committee on Causes and Prevention of Highway Accidents..........Dr. Knight Dunlap, *Chairman*
Professor of Experimental Psychology, Johns Hopkins University

Report of Committee on Mineral Aggregates..........W. J. Emmons, *Chairman*
Director of Highway Laboratory, Michigan State Highway Department

ADDRESS OF WELCOME

Albert L. Barrows

Assistant Secretary, National Research Council

Mr. Chairman, Members of the Highway Research Board and members of this meeting, I am here this morning to express to you the great regret of Dr. Kellogg, the Permanent Secretary of the National Research Council, over not being able to greet you as has been his wont at the preceding meetings of the Highway Research Board. Dr. Kellogg, however, wishes me to extend to you the very cordial greeting of the Council and to assure you of welcome in holding these sessions here this week.

This is the 9th annual meeting of the Highway Research Board. To have maintained this series of meetings for the annual clearing of the latest information on the building of roads is in itself an important contribution to the science of road building. But these are after all your meetings—the sessions of engineers of highway construction for the discussion of the best ways to carry on their work. The National Research Council is happy to have been able to join with you in making this series of meetings possible.

The purpose of the Highway Research Board of the Council is the encouragement of research upon the building of roads. With the ever increasing amounts of money expended on our highway systems now reaching figures surpassing most of the other enterprises on which men are engaged, it well behooves us to ascertain with care the right way in which to lay down these highways so that there may be gained the most effective results from this huge expenditure. The impressive thing about this whole great enterprise is that of these large sums expended it requires but a comparatively small fraction to find out how best to expend them.

I wish to compliment the Board, at this time, particularly on having prepared a comprehensive plan for highway research on a national scale. This is to be presented to you at this meeting for discussion and for your suggestions. The resulting program for coordinated investigations upon the diverse problems of the building and management of highways, including as a most important item the matter of safety to human life in our use of this system of transportation, should be an indispensable directive in the efficient carrying out of your purposes and of those of the Highway Research Board with you. In these discussions, in the formulation of these plans and in the successful carrying out of these plans, you have the very best wishes of the National Research Council. The Council welcomes you to this meeting and is very glad indeed to be able to have some part with you in this important work.

RESPONSE TO ADDRESS OF WELCOME

F. H. Eno

Chairman, Highway Research Board

Dr. Barrows, in response I wish to thank you for your address of welcome to us, to this very beautiful building. We are pleased to come here. I hear the members speaking of the beauty of the building and other attractions. I want to say that in the past nine years, as I look back over the sessions of this Board, we have come here usually expecting something. We hope this year we have brought to this meeting a rich harvest that will be of interest and value to the members and to the community at large.

As I review the past nine years I see we have come no mean distance on the problem of highway research in the betterment of roads. Our mileage of roads has increased, and the value and efficiency of the road and its durability have increased during these past nine years. We cannot lay our fingers upon specific things always and say to the world that this was brought about by the Highway Research Board and that this has saved so many miles and so many thousands of dollars to the nation and the states, but viewing it by and large I think there is no question in the minds of those engaged in this research that benefit has come in dollars and cents and in large sums to the states and to the nation as a result of the meeting together of these men who are members of this Research Board.

We will look back to the first session. I remember the result was an outline of research made up at that time—of course naturally limited because of our inexperience—but a list of 479 projects was presented by the chairman in that report, coming from 132 different organizations throughout the United States, of research that had just been finished or was in the process of being worked upon. A large number of those 132 organizations were made up of the state highway departments and the universities. They were joined in this by other organizations but these were the two groups that had the most projects in process of work.

Then came the second session and at this meeting we had reports of the tests at the Arlington experimental plant. We had the reports, not complete but partially complete at least, of the Bates road, and of the Pittsburg tests in California. They brought to us the fact that we must change the method of our construction of concrete pavements to thickened edges. That has brought undoubted saving to every state in this Union. Year by year since then we have come, bringing first one thing then another. There are the reports on the wind resistance to vehicles, tire wear, cost of vehicle operation on a mile of road, the question of not being able to freeze water when you want to freeze it in certain soils, and the ability to freeze it in others, the impact tests on concrete, traffic analysis, grading of top soil mixtures, and a whole line of these reports that have been coming in in the last years; also two special reports, "Economic Value of Reinforced Concrete" issued as a part of the 5th Annual Report, and "Low Cost Improved Roads," issued as a part of the 7th Annual Report. Let me say here that I believe we are now turning our attention more strongly and more intensely to the question of how we can economically develop the low cost roads for we have come to a time when our agricultural population has been taxed to its limit and is not yet out of the mud. We have two million miles of roads or more that must still be improved in some way, and we must do it as cheaply as possible. The "Low Cost Improved Roads" report is opening the way.

As I considered the work of this Board a vision came to me with this superscription, "Research, a Power for Peace, Prosperity and Happiness of Humanity."

My mind flashed back to the Scriptural story of the tower of Babel, and I saw humanity speaking one language, and because they could understand each other, and each other's hopes and feelings, they aspired to pull together and build a tower even unto heaven.

Then through some psychological cataclysm, humanity was scattered to the four winds. The absence of communication soon produced provincialism and this grew to separate and distinct customs, habits and languages. Fear of subjugation, commercial competition, seas and mountain ranges have proven barriers to the brotherhood of mankind. Finally intercommunication began once more, extremely difficult and slow at first but gathering momentum slowly. The first great step was the use of sailing vessels. Then about 125 years ago the invention of the steam engine and its application to steamships and railroads widened contacts and brought people into association and competition, but because of past isolation it did not always prove a peaceful contact.

Again in 1837 the invention of the telegraph, and in 1876 the invention of the telephone added human contacts and made races better acquainted with each other.

It is within the past 25 years that research has added three agents that have begun and will continue to revolutionize civilization, namely, the motor car, the motor plane and the radio. The world has shrunken to a football. We eat our toast this morning, and listen to the words of the lords of England, or the complaints of the rajahs of India, or the night's defeats of the armies in China, or Byrd's yesterday's flight across the Antarctic. The slowest of these three civilizing, humanizing agents is the motor car because it waits upon the slow and tedious process of building roads across marshes, over mountains and through deserts. The cost is the greatest also, but the contacts formed by it are more intimate and individual.

The roads of Ohio, Indiana, Kentucky, and of Massachusetts, New York and Pennsylvania are wiping out the provincialism of Boston and of the mountains of Kentucky. As we learn each other's language, as we eat salt together from a common table, as we know each other better; we will grow less suspicious; we will appreciate each other's good points more and realize that the bad points are not so sharp. The friction of our racial cogs grows less and peace with its concomitant prosperity will come apace.

The first great chapter has been written in Pan Americanism, the next chapter will be Pan racialism.

Tennyson's prophecy has come true:

> "Men, my brothers, men the workers, ever reaping something new;
> That which they have done but earnest of the things that they shall do;
>
> For I dipt into the future, far as human eye could see,
> Saw the vision of the world, and all the wonder that would be;
>
> Saw the heavens fill with commerce, argosies of magic sails,
> Pilots of the purple twilight, dropping down with costly bales;
>
> Heard the heavens fill with shouting, and there rain'd a ghastly dew
> From the nations' airy navies grappling in the central blue;
>
> Far along the world-wide whisper of the south-wind rushing warm,
> With the standards of the peoples plunging thro' the thunder-storm;
>
> Till the war-drum throbb'd no longer, and the battle-flags were furl'd
> In the Parliament of man, the Federation of the world."

A PROPOSED PROGRAM OF HIGHWAY RESEARCH

INTRODUCTION

Thomas H. MacDonald, *Chief*

United States Bureau of Public Roads

Mr. Chairman, what I say will be largely introductory to the presentation of a plan for a comprehensive research program by the Director of the Highway Research Board, Mr. Crum. I hope it will not be wasteful of your time to take a few minutes to review something of the thought, purpose, and underlying conceptions back of the establishment of the Highway Research Board.

The conception of this Board primarily, it seems to me, is to assist the individual who is engaged in research. We have regarded the matter of research as being somewhat of an abstract matter in which the individual does not count. In my thinking, more and more helpfulness to the individual has come to be the very important objective of such meetings and such activities as the Highway Research Board is carrying forward, and I have attempted to outline some of the underlying conceptions of the purpose of the Board.

First: To provide a particular and exclusive opportunity for the research worker to express his ideas and ideals to the public and to his fellow workers.

Second: To develop the general conception and promote the acceptance of the dignity and importance of research in the highway and related fields.

Third: To lend encouragement and to help sustain enthusiasm in the necessarily long continued efforts of the individual. That is, I can conceive of a man carrying on work, feeling his way along an untrodden path, being occasionally discouraged as to whether he is making progress toward his objective or not. To any such individual the benefits that can come out of a meeting of this kind are the enthusiasm and stimulation that come from contact with men who may even be in the same uncertain state of mind.

Fourth: To promote friendliness and a more generous appreciation of results secured through efficient and concentrated attention to the development of new knowledge.

Fifth: To bring about through mutual confidence and unselfish desires to serve the public interest, a pooling of resources, of data and new knowledge, and the organization and correlation of efforts to make highway research more effective, efficient and conclusive.

As you know the charter for the establishment of the National Research Council came by request from President Wilson to the National Academy of Sciences. In 1918, President Wilson made a further request to continue the Council and issued at that time an Executive Order directing all governmental agencies to cooperate with the National Research Council. That gives every governmental department executive authority to cooperate. The Highway Research Board was formed under authority of the Council. The National Research Council carries authority and dignity which can well be recognized by all interested agencies in undertaking to cooperate in the work and to devote funds to the purposes outlined by the Highway Research Board. We have had, since the establishment of the Highway Research Board, the cooperation of the Council in every effective manner within its power and disposal. It is proper for me at this time to express appreciation for the attitude taken by the directing officials of the National Research Council in assisting the Highway Research Board in so many ways.

There can be very little question as to the importance of research in this highway field, although there is a very large misuse of the term " research." It comes to me over and over as I go through engineering literature on this particular subject that there is a great paucity of new information upon the various subjects connected with highway engineering and utilization; that we find little new data that some research worker has contributed. Existing data are repeated, revamped, and reworked into various other forms, but it is the same old information dished up in a new form. We cannot rest content with the amount of research information that is actually being developed. Compilations are not research. They may be the necessary preliminary preparation for the research worker. He ought to know what has gone on in the field before, but research to me means the searching out of new and important information. All research information is important because all new information is important. It may be positive or it may be negative but nevertheless it is important.

Quoting from the last report of the Secretary of Agriculture in which he discusses the question of how nearly we can measure the results of research in a material way, he says, " Scientific investigations are sometimes most fruitful when directed merely toward the discovery of fundamental principles."

The important problems in the highway field have been so insistent that there has been the tendency for the highway administrator to demand from his technical staff answers to questions which they have not had the opportunity to carry to final completion. He has had to make an answer. We have only half answers to many problems, but they are admittedly better than no answer to the administrative official carrying on a large program. As highway administrators we have necessarily accepted less scientific information than we knew we needed to more adequately insure the expenditure of large public funds. The highway administrators of the country must be brought to see that a reasonable percentage of the funds for highway purposes invested in research will in the end produce far greater results than to continue without developing new methods and new processes through research. The idea must be accepted by the highway administrators and public officials of the country who are handling funds that may be used for research, that the expenditure for this purpose is not a living expense. It is not an expenditure that is thrown to the four winds, but rather an investment that will return to the public profits a thousand times over.

There are many important problems in this highway field. The expenditures are mounting but the service demands are increasing more rapidly. You will notice I have not totaled the expenditures for highway purposes. These are not so important relatively since we have more than 26 million automobiles and other motor vehicles operating on the public highways. The amount we spend to give them a road bed over which to operate is not a very important percentage of the whole cost, although it may mount into large figures as a total. The earning capacity of improved highways has been proven. There is no question about that and, while it is true that our expenditures have been mounting, these mounting expenditures have been largely paid by the man who uses the highway. Probably expenditures will increase as long as our operation of motor vehicles continues to increase. The automobile owner will not be discouraged by the taxes he is paying and the increased funds expended if he obtains continuously better highways to use.

A few of the problems which I think are most important now are the following:

Soils and their characteristics, particularly as subgrades;

Low cost roads of adequate service character;

> In talking of low cost roads, we should add the qualification that they be of adequate nature to carry the traffic.

Highway transport utilization and attendant problems;

The planning of highway systems;

> There is one fact we have to face in this country. We have too many highways, particularly in the big agricultural states. There are about three million miles of public highways and to assume that we can rapidly improve any considerable percentage of this whole mileage is to disregard the facts. It is an unnecessarily large undertaking and we must reduce our mileage of highways to be improved for general use. Rather than bring back marginal lands into production, we had better turn back some of our rights of way that are fenced off and used for little other than growing weeds. The planning of highway systems becomes one of the crying needs, particularly in our big states that have so many miles of roads that they cannot hope for many years to improve the major percentage of the mileage.

The financing of highways is another subject which must receive attention; also

The effects of heavy vehicles, and relative taxation or other charges for highway purposes.

The highway departments in the states are leaders in determining policies. The highway administrator is dependent largely upon the work of his technical staff to secure the information on the various problems which confront him, so that in the end the efficient research worker has an important position and the work that he does has much weight in determining policies which are put into effect. The research worker has in that way a direct access and a direct control of the shaping of policies which determine the progress which we shall make in this public work in the United States. I cannot over-emphasize the importance which I place upon the research staff within the highway organizations of this country where they are capable and where they are supported with sufficient funds actually to produce results.

Effective research, however, in my judgment, can only be secured by a national research program, rightly coordinated. On an important subject recently, out of the available 36 research projects from which to gain information, there was not the coordination sufficient to draw definite conclusions, so coordination is as important as doing the work itself. We must obtain comparable results and when we have 36 research projects upon which to draw for information we must be able to produce information the administrator can use. If he cannot, it is our own fault if we do not get the funds necessary to carry on research.

This matter of coordinating the research program is most important since we will get the weight of all the research that is being done behind the research of the individual worker. So long as we overlook any uncontrolled variables of research projects, the administrator is not justified in giving full weight in determining his policies. The results may be open to question in some essential respect. This is why research coordination is most important.

In closing may I quote again from the report of the Secretary of Agriculture:

> " The ultimate justification in utility may be remote rather than immediate. Results will come, which cannot be reckoned in advance. Results achieved as an unexpected by-product are often more important than those originally contemplated."

To achieve the greatest success, to be able to record continuous real progress and incidentally to advance the interests of the whole highway research fraternity, we must put our house in order. There is no more necessary or effective procedure than to unite on a national research program.

REPORT OF COMMITTEE ON COORDINATION AND PROGRAM

R. W. Crum, *Chairman*

A PROGRAM OF HIGHWAY RESEARCH

Since the advent of the motor vehicle and the extraordinary development of its use, traffic conditions have been changing so rapidly, that new problems and new aspects of old problems are constantly arising.

The result has been, and no doubt the condition will continue, that traffic needs must be cared for as well as possible with what knowledge is available at the same time that correct principles are being developed for future use.

Highway Research has already contributed materially toward meeting modern conditions of highway transportation. However, many important questions are yet unanswered and much thorough and painstaking research will be needed for their solution.

It is the purpose of the Highway Research Board to set up a comprehensive program looking toward the solution of these problems. Such a program will be subject to continual change. Final answers will be reached at times, new projects must be frequently added and changing conditions will no doubt render some work valueless even before it is completed. Having a program outlined it is our hope to keep it up to date continuously.

Such a research program as this will not be of much value unless the necessary experimental work is accomplished, the results thoroughly studied and sound conclusions drawn and put into practice. The Highway Research Board therefore plans to do much more than merely outline a research program. The plan of activity may be briefly stated as follows:

1. To draw up a comprehensive program of highway research.
2. To compile and have available for dissemination, information and references on the existing status of knowledge of the various projects.
3. To compile and keep up to date lists of the current research projects throughout the United States.
4. To promote with the various research agencies the carrying on of the experimental work recommended in the program.

5. To conduct special investigations of national scope.
6. To correlate the work of the various investigators, to study the results of research work as they become available, and to draw sound and usable conclusions wherever possible.
7. To disseminate research information.

To accomplish these objectives will require the co-ordinated efforts of the officers and staff of the Board, technical committees of unquestioned standing and contact men with all of the research agencies and highway departments.

We are presenting herewith statements and analyses of the highway problems as they exist today with suggestions as to needed research in their solution. It is not intended that the items of needed research listed shall be exhaustive, but rather that they shall be indicative of the field that should be covered, and that they may serve as a starting point for extension and sub-division by the various committees of the Board. It is expected that the various committees will expand this outline into complete research programs covering their various interests.

OUTLINE FOR A PROGRAM OF HIGHWAY RESEARCH

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I. Highway Administration and Finance

ADMINISTRATION

The problems of highway administration have to do principally with the allocation of the available funds, decision as to priority in road improvement programs and location as respects termini and general route. Administration includes matters of policy in handling the funds available for road work.

There are forty-eight systems of state highway administration in the United States, each differing from all of the others in many important particulars. The administration of the secondary road funds by smaller governmental units also varies widely in method.

Published data are not available upon which decision could be based as to the method of highway administration likely to be most efficient in any given case.

Although state highway administrative organizations all differ in detail, they may be grouped into three general systems.

1. An appointive commission, with an executive officer appointed by the commission. The chief engineer is sometimes the principal executive officer.
2. A director or superintendent appointed by the governor, who handles both matters of policy and executive direction.
3. A director or superintendent elected by the people, who handles both matters of policy and executive direction.

The success of any administrative organization depends upon the abilities and personalities of the men at its head, and there are outstanding samples of well managed state highway departments under each of these categories. However, it is possible that thorough inquiry might reveal some inherent advantages or defects in one or more of these systems. Although such an investigation would be costly and difficult to make, it should be included in a comprehensive program of highway research. The fruits of such an inquiry would be of great value to highway officials in dealing with legislative bodies, and to progressive legislators themselves in framing adequate highway laws.

FINANCE

The general problem is the equitable distribution of the cost of providing highway service in proportion to the benefits derived therefrom. It may be assumed that benefits accrue in four ways, (1) National benefit by providing for free flow of traffic between states and by provision for national defence, (2) General benefit to community life, (3) Special benefit to abutting property, and (4) Benefit to the individual user of the roads. Revenues should be raised from each of these four sources, the unanswered questions being: What portion should each source contribute, and in what ratio should the funds be allocated among—the through traffic highways, the community roads, the purely local roads, and municipal streets?

1. FEDERAL AID

The principle of federal aid for the improvement of roads of national importance is firmly established on a sound basis. The amounts and allocation provided by acts of Congress have had a stimulating effect on road improvements, and a profound beneficial effect upon the standards of road construction. Federal aid should be in proportion to the national benefit derived from adequate road systems.

2. GENERAL TAXATION

To what extent should the general public contribute through property taxes, and what proportion should be expended upon each of the three classes of roads . . . . (a) through traffic highways, (b) community roads, (c) purely local roads? In other words to what extent should property taxes be levied in the interests of each of the three classes of roads? On the one extreme we have the local road of importance only to the residents along the road, used only by them or by those who seek access to them. On the other extreme we have the through highway used by everyone in the community as well as by others from outside passing through. To what extent does the improvement of each of the three classes of roads benefit the entire community irrespective of the actual use of the roads?

3. ASSESSMENT AGAINST ABUTTING PROPERTY

Agricultural property is undoubtedly benefited by proximity to a good road over which its produce can be marketed and also to some degree by the effect of the road improvement on its desirability as a home site. The question is, at what stage in road improvement does the special benefit cease to increase? These benefits can be measured by the increase in selling price of the land caused by the road improvement, and in the absence of such cost data by farm records as to cost of marketing and savings due to timeliness in marketing.

4. MOTOR VEHICLE TAXES

Taxes imposed upon vehicles are of three kinds at the present time: (1) General property taxes; (2) license and registration fees; and (3) taxes on gasoline. It may be assumed that the most equitable taxes on the vehicle are those based upon the amount of use of the road and upon the destructive effect of the vehicle upon the road. The tax on gasoline is the best measure yet proposed for a tax based upon use of the road. The destructive effects of different types of vehicles have not yet been evaluated. Information is needed as to the costs of vehicle operation and as to the costs of providing highway service adequate for different classes of traffic. For instance, a 6-inch concrete slab might be heavy enough for a large part of the traffic on a certain highway, but the presence of bus and truck traffic might make necessary an 8-inch slab. In such a case a tax rate based on mileage alone might not distribute the burden equitably. On the other hand it might not be equitable to saddle all of the cost of the extra two inches of concrete upon the busses and trucks because the general public benefits from the ability of the busses and trucks to operate over the road.

Another problem of immediate importance is the distribution of the proceeds of gas or other vehicle taxes, as between the various classes of rural roads and city streets. If such taxes are a measure of road use by the vehicles, all of the roads used have some claim upon the proceeds of the tax, but as yet no data are available showing what the relative amounts should be. The proportion of a gasoline tax, for instance, that should go to the various classes of roads, and to the city streets will depend upon the relative amount of traffic and upon the proportion of the cost of each class that should be borne by the road user.

5. USE OF CREDIT

If roads are constructed by means of bond issues, they can be put into service sooner, but the earlier expenditure of capital will mean additional interest charges which will increase the net annual cost of providing the highway service. The question is whether or not the savings in total highway transportation costs during the extra years of service will compensate for the additional roadway cost. The ability to find the answer in any given case will depend upon the available knowledge of costs of transportation over various types of road surfaces and upon the amount of traffic.

NEEDED RESEARCH

Administration and Finance

ADMINISTRATION

1. A critical study of results secured under various methods of state highway administration.
2. A critical study of results secured under various methods of administering secondary road funds by counties and townships.
3. A critical study of results secured under various methods of municipal highway administration.

FEDERAL AID

1. Study of trends in inter-state highway traffic.

GENERAL TAXATION

1. Studies of economic changes in typical communities to show the benefit of road development in the life of the community.
2. Studies in typical communities to show the stage of road improvement at which the benefits to the road user begin to over-shadow the general benefits. For instance, a low type of surfacing might be sufficient for general community needs, although a high type may be needed to care for the traffic adequately.

ASSESSMENT AGAINST BENEFITED PROPERTY

1. Studies in a number of scattered communities of the effects of different types of road improvement upon land values and marketing conditions, and the effect which distance from marketing centers has upon land values, and upon the type of road necessary for farm to market service.

MOTOR VEHICLE TAXES

1. Vehicle operating costs, for various types of vehicles, on various kinds of road surfaces.
2. Costs of providing highway service for various kinds of traffic.

(Note.—These (1 and 2) will be further discussed under Highway Transportation Costs.)

3. Relative use of different classes of roads.—Traffic surveys on all classes.
4. Under what conditions are toll roads and bridges justified?

USE OF CREDIT

1. Data on highway transportation costs.

(NOTE.—To be further discussed under Highway Transportation Costs.)

2. Several states have already expended the proceeds of large bond issues for road improvements. Studies of economic conditions in those states as related to highway transportation should go far to answer the question as to the practical value of thus speeding up the road building program.

II. HIGHWAY TRANSPORTATION

COSTS

The annual cost of transportation over any length of highway consists of the yearly cost of providing the roadway ready for service plus the cost of operation of all vehicles while using that particular road during the year.

A knowledge of all of the factors entering into this total cost is needed to furnish a basis, for equitable taxation of vehicles, for the proper layout of highway improvement programs, and for the economic design of the roads. In other words, *highway transportation cost* is a dominant factor in the solution of all of the major highway problems.

If the vehicle owners are to be expected to pay a large part of the highway service bill, they should be taxed in proportion to the extent to which each uses the roads, and in proportion to the effect of his vehicle upon the annual cost of providing the highway. The class of road service to be provided to the user should also be taken into account in fixing the tax rate, for the user can afford to pay more for the use of a good road providing a free flow for traffic, than for the use of a poor road with frequent constrictions. Methods for accurate determination of these factors depend upon further study of costs.

Vehicle operating costs are affected by the type of road surface. In planning an improvement program the type of improvement should be used for which the annual transportation cost will be the lowest at which the expected traffic can be adequately served.

In designing the road itself, consideration should be given to the effects of such features as grade, alignment, cross section and prospective increased use upon the total transportation cost. Knowledge of the effects of such features upon such items as gasoline and oil consumption, tractive resistance, and upon schedule speeds is needed for economical designing.

The principal elements of highway transportation costs are as follows:

1. Roadway costs.
    - A. Capital costs, consisting of interest on the following investment items.
        1. Right-of-way.
        2. Primary construction.
        3. Surfacing.
        4. Engineering and supervision.
    - B. Upkeep costs, consisting of the costs of the following items plus interest thereon.
        1. Maintenance of road surface.
        2. Maintenance of right-of-way.
        3. Administration.

The principal elements upon which data are lacking in connection with roadway costs are the effects of quantity and kind of traffic upon maintenance costs, and the rate of depreciation and salvage value of various types of roads under various conditions.

2. Vehicle operating cost.
    - A. Capital costs.
        1. Interest on initial cost minus cost of tires and tubes.
        2. Depreciation.
    - B. Operation Costs.
        1. Tires and tubes.
        2. Gasoline and oil.
        3. Maintenance.
        4. Driver's pay.
        5. Depreciation (one-half considered an operating cost).
        6. License fee.
        7. Garage.
        8. Gasoline tax.
        9. Insurance.

More data are needed on all of the items of vehicle costs that are affected by mileage, for those are the items that are also affected by the roadway characteristics. Also data are needed on the cost of such items as waiting on traffic lights, and slow speed due to congestion, especially in the case of commercial transport vehicles.

UTILIZATION OF HIGHWAYS

A most important factor in the proper development of highway systems is the extent of productive use of the highways. The extent of movement of people and commodities over the highways has important bearings, not only upon the economic development of the highways, but upon the future of other forms of transportation as well. Certainly continuing study should be made of the trends in highway utilization. One problem is to determine the conditions under which it is economical to move passengers and commodities over the high ways. Its solution will involve detailed investigation of such matters as transportation cost per item moved, time saved or lost, condition of shipments, convenience of terminals, etc.

NEEDED RESEARCH

Transportation

ROADWAY COSTS

1. Collection of data on relation of maintenance costs to traffic for all types of surfaces. Promotion of the use of uniform records of cost and traffic.
2. Collection of data on cost of resurfacing and on salvage value.
3. Study of rates of depreciation.

VEHICLE OPERATING COSTS

1. Collection of data on vehicle operating costs, especially on busses and trucks.
2. Study of factors, outside of vehicle design that affect operating costs.
   - (a) Road surface (tractive resistance).
   - (b) Grade and alignment.
   - (c) Tire wear.
   - (d) Car maintenance (mechanical depreciation).
   - (e) Effect of time lost, due to enforced waits and congestion.

UTILIZATION OF HIGHWAYS

1. Collection and study of data on amount of commercial transportation of passengers and commodities over highways.
2. Study of movement of agricultural produce and live stock to market centers by highway, including costs, condition, convenience, time factor, etc.

3. Study of movement of manufactured articles and raw materials by highways, including costs, condition, convenience, time factor, etc.

III. Highway Design

The design of the highway includes roadway, drainage structures and surface. The general problem is, how best to build for maximum economy and safety. The principal consideration in dealing with each item, outside of safety, is that of cost. By cost is meant the total transportation cost based on both roadway and vehicle operating costs. Traffic has now become so great that sound economic designs can no longer be based on roadway costs alone. Costs of vehicle operation, and savings to the user by providing for unrestricted flow of traffic must be taken into account. In fact, on heavy traveled roads the vehicular costs entirely overshadow the mere cost of building the roadway.

ROADWAY

Location, grades, alignment and width have a large effect upon transportation costs, traffic capacity, and safety. Their effects upon vehicle costs are measured by tractive resistance, both rolling and wind resistance, fuel consumption, mileage, ability to maintain schedule speeds, and traffic capacity. Their effects upon safety must be evaluated from accident statistics. Their effects on roadway costs, outside of construction, are measured by the cost of maintenance.

The principal determining factors in establishing these features of a highway are, quantity and kind of traffic and considerations of public safety. Unusual conditions caused by such phenomena as landslides, subsistence, etc., are troublesome in some regions.

DRAINAGE STRUCTURES

Bridges and culverts do not affect the cost of vehicle operation unless they are restricted in width and their cost is not ordinarily a large proportion of the roadway cost. Therefore, on heavily traveled roads, where the vehicular costs are the principal item, any reasonable expenditure to secure attractive structures that are adequate, durable and safe can be justified. The general problem is to build in each location the structure that will adequately meet the needs of the situation from both utilitarian and æsthetic standpoints.

BRIDGES

Probably the most difficult problems in bridge design are the selection of type and size. Type depends upon relative economy and upon appearance. The size of waterway depends upon the size and character of drainage area, and rainfall to be expected. Detail design depends upon static and impact loads and upon the strength of materials. With respect to type and size the principal field for research work lies in the study of relative economy and durability of different materials, and in collecting run-off data.

With respect to detail design the field for research is in the development of new materials and new combinations of materials, and further studies of loads due to impact.

CULVERTS

The selection of type of culvert depends upon relative economy. Detail design depends upon the size of waterway needed and upon the loads to be supported. The relative durability of different types of culverts for a given set of conditions should be evaluated by the annual cost, taking into account interest on investment, maintenance, depreciation and salvage value. Information is lacking at the present time on depreciation and salvage values. The effects of loads on culvert pipe have been extensively studied, and general principles are well developed.

SUB-DRAINS AND ACCESSORIES

Attempted stabilization of roadways by under drainage has long been practiced, but without any very definite guiding principle as to the conditions under which it is effective and worth the cost.

SURFACE—SUBGRADE, BASE COURSE, WEARING COURSE

As in most other phases of highway work, the general problem is to provide the surface that will take care of the traffic satisfactorily at the least cost for highway transportation. The choice of type and design of surface are affected by the character of the subgrade, the amount and kind of traffic, climate and topography, the annual cost of various types of surface and safety precautions.

The following introduction to an article by Hogentogler and Terzaghi on "Inter-relationships of Load, Road, and Subgrade" in Public Roads, May, 1929, may well serve also as to statement of objectives in research on design of road surfaces.

"It is intended to point out how a knowledge of the primary subgrade soil characteristics considered with that of the different pavement properties may prevent the engineer from specifying one type of pavement when conditions will not permit of its adequate construction or operation; from increasing pavement thickness in an attempt to eliminate troubles not caused by lack of thickness; from attempting to cure structural defects by improving the wearing course; from attempting to prevent abrasive troubles by improving the structural resistance; from attempting to estimate the comparable service values of rigid and non-rigid pavements for conditions which clearly indicate the necessity for pavements furnishing high load distribution; from supplying only cohesion when friction is also required and vice versa; and from improving the subgrade to prevent troubles resulting from improper design, construction or maintenance of the pavement."

Subgrade—Effects on Design of Surface

The factors, according to Hogentogler and Terzaghi, involved in the study of the relationship between subgrade soil and pavements are as follows:

"Subgrade properties due primarily to the raw constituents of the soil, also those due to the field conditions under which the soil exists; pavement conditions caused entirely by the subgrade support, and conditions only partially caused by the characteristics of the subgrade; individual conditions of subgrade support which produce particular conditions in the pavement; the relative influence which similar subgrade conditions exert upon pavements different in type; the soil characteristics disclosed by a particular group of laboratory test results; the individual raw constituents which cause the soil to exhibit particular physical properties; the relative influence which the arbitrary test procedure and the physical characteristics of the raw constituents of soils exert upon the results furnished by laboratory tests; and the extent to which the physical manifestations disclosed by the laboratory tests serve to explain the behavior of soils when subjected to field conditions.

"Pavement behavior may depend upon the character of the subgrade soil material (raw constituents), upon the structure of the soil in its natural state (dense or loose, homogeneous or full of cracks and root holes), upon the soil profile (variation in depth of the different soil zones and the relative occurrence of permeable and impermeable strata), upon adjacent topography (through its influence upon the occurrence of surface and underground water), upon climatic conditions (well distributed or intermittent occurrence of rainfall and presence or absence of frost action), or upon any combination of these variables."

Traffic—Effects on Design of Surface

The effects of the quantity, kind and weight of traffic are the dominant factors in the design of the road surface since the only reason for building the road is to serve the vehicles that use it. The type of surface should be the one which will accommodate the expected traffic at the least transportation cost. For instance, a low cost road with high maintenance costs may be more economical for a light traffic road than a more expensive but more durable type, because in such a case the total saving in operating costs to the small number of vehicles using the road, by having a higher type road would not be large enough to overcome the high annual charge for interest due to the greater first cost of the high type road, and conversely, a high type will usually be found to be more economical for the use of heavy traffic.

The physical effects of the traffic upon the surface are caused by speed, weight, and impact, and are ultimately measured in terms of cost of upkeep.

A difficult problem in each specific case is the determination of the controlling weight of traffic unit upon which to base the design. Information as to the frequency of use by the heavier vehicles is important.

Climate and Topography—Effects on Design of Surface

Some road surface deterioration is caused by action of the elements. Rainfall and extremes of temperature in some parts of the country are determining factors in design. The principal effect of ordinary topographical conditions upon structural design is the provision that must be made for surface water run off.

Road Surface Types

Road surfaces may be classified as high, intermediate, and low type according to their suitability for heavy traffic main thoroughfares, medium traffic thoroughfares and community roads, and light traffic local roads. The principal problems in connection with all of them, have to do with the development of the necessary characteristics to handle the traffic satisfactorily at minimum transportation cost.

The high types at present of importance on rural highways are concrete, asphalt, and brick.

High Type Surfaces. (a) *Concrete and Concrete Bases.* The problems involved are the determination of stresses caused by static loads, impact loads, volume change and fatigue, and methods of rational design to withstand the stresses economically.

Problems concerned with the quality of concrete will be considered under materials.

(b) *Sheet Asphalt and Asphaltic Concrete Surfaces.* The problem is to construct durable surfaces economically, that will not be displaced under traffic.

(c) *Brick Surfaces.* The principal remaining problems have to do with the size of the brick and the character of the joint filler to meet conditions in various parts of the country.

Intermediate Type Surfaces. The intermediate types of surfaces include: Waterbound macadam, traffic bound surfaces of various materials, bituminous macadam and bituminous bound surfaces. The problem is largely one of economy as the principal objective is to build roads that can be maintained in a satisfactory condition at reasonable cost for highway transportation. The relation between amount and kind of traffic and maintenance cost is especially important on these roads. From a structural standpoint the problems have to do with the effects of climate, and traffic upon durability. Much of the needed research has to do with the materials, which will be considered in another section.

Development of road surfaces of this class will be largely a matter of experimentation in service. More detailed reports of such experiments should be made.

Low Type Surfaces. The low type surfaces include dirt, top soil, sand-clay, etc. The problem is to provide a road at very slight cost that will furnish approximately all weather service for light traffic. Top soil and sand-clay where available have been found reasonably satisfactory. The problem is very pressing in the great agricultural regions where the cost of hauling surfacing material is prohibitive for these roads.

" While this problem is so simply stated, this statement does not itself carry evidence of its importance. There is probably no other single investigation that can be defined that is of as much importance in the highway research field. What is sought is the development of some now unknown method of soil treatment that will result in effective and very low cost improvement of the thousands of miles of earth roads."

NEEDED RESEARCH

Design

ROADWAY

For suggested researches bearing upon roadway see Transportation—Maintenance—and Traffic.

1. Data on landslides.
2. Data on subsistence.
3. Method for building fills in marshes.
4. Relationships between highway design and speed, safety and traffic capacity.
5. Relation of width, crown, curvature, sight distance, ditches, shoulders, etc., to accidents.
6. Relation of curvature, super-elevation, width, to speed and traffic capacity.
7. Effect of constrictions on capacity and accidents.
8. Effect of road condition, visibility distances, etc., on formation of " cues," and possible means for their elimination.

DRAINAGE STRUCTURES

Bridges

1. Conditions required for durability of concrete structures.
2. Protection of steel work to secure maximum durability.
3. Collection of run-off data.
4. Measurement of stresses due to impact.
5. Allowable unit stresses in concrete and reinforcing steel.
6. Suitability of various materials and designs for floors.

Culverts

1. Development of methods for determining the economic value of various types of small drainage structures. The principal unknown items are rate of depreciation and salvage value.

2. Determination of economic value of various types of culverts under various conditions of use, such as height of embankment, character of soil, erosive and corrosive effects of water carried, etc.

Sub-Drains

1. Further study of the interrelationships between soil characteristics, climate, topography, under drains and road characteristics.

SURFACE

Subgrade

1. Continued development of laboratory tests to aid in classification and identification of soils.
2. Relationships between laboratory tests and properties and behavior of soils in roads.
3. Study of the behavior of soils under various moisture conditions.
4. Effects of surface treatments of subgrades.
5. Effects of subgrade treatments on supporting quality.
6. Surveys of relationships between soil characteristics and road surfacing behavior.

Effects of Traffic on Design of Surface

1. Wheel load intensities.
2. Distribution of wheel loads through various surfaces.
3. Effect of volume and weight of traffic on maintenance costs of various surface types.
4. Effect of speed on road surfaces.

Effects of Climate and Topography

1. Data on run-off on various grades under various intensities of rainfall.

Concrete Surfaces and Bases

1. Study of fatigue of concrete.
2. Study of the balance of various thickened-edge cross-sectional designs.
3. Study of longitudinal joints, with reference to edge strengthening.

4. Causes for blow-ups.
5. Theoretical study of various influences upon stresses.
6. Experiments to determine values of the modulus of subgrade reaction.
7. Study of volume changes under various conditions. Effects of moisture and temperature on volume changes.
8. Methods for constructing joints and planes of weakness.
9. Effect of reinforcement.
10. Strength of concrete and thickness necessary in bases for use with asphalt and brick wearing surfaces.

Sheet Asphalt and Asphaltic Concrete Surfaces

1. Methods of maintenance to renew volatile constituents.
2. Study of conditions affecting stability.
3. Development of test for stability of mixtures containing coarse aggregate.
4. Study of finishing methods to provide smooth surfaces.
5. Study of suitability of flexible bases.

Brick Surfaces

1. Service tests of various thicknesses of brick.
2. Characteristics of filler under various climatic conditions.

Intermediate Type Surfaces

1. Data on effects of weather and climate on various combinations of materials.
2. Data on effects of traffic.
3. Rate of wear of waterbound and traffic bound roads and effect of dust prevention measures thereon.
4. Salvage value of such surfaces in connection with change to higher type.
5. Critical study of various types of surfaces now in use.
6. Design of cross section for bituminous macadam.

Low Type Surfaces

1. Investigation of possible methods of stabilizing soil to form a road surface.

IV. MATERIALS AND CONSTRUCTION

MATERIALS

The two factors that are of most importance are the effects of the characteristics of materials upon results and the identification of characteristics by test methods. Knowledge of both is necessary for proper design and use. The general problem with respect to materials is the use of the most economically available material in proper combinations to yield the result expected by the designer of the pavement or structure. Conversely good engineering requires the design of structures in such a way as to utilize with economy the most economically available materials. There are yet many factors as respects highway materials that need investigation.

CONSTRUCTION

Construction methods and appliances are usually developed as the result of trial, rather than by premeditated research. However, research often becomes necessary to evaluate the results secured by new methods or appliances, in cases where the quality of result is not obvious. It is not possible to lay out in advance a program of such research work. A place on the program should be provided for the inclusion of such projects whenever they appear.

NEEDED RESEARCH

MATERIALS AND CONSTRUCTION

MATERIALS

Concrete Materials.

Cement.

1. Development of more determinative tests than are now available.
2. Possibilities of high early strength cement.

Aggregates.

1. Method of test for concrete making quality of fine aggregate to replace the mortar strength test.
2. Studies of effects of such characteristics as shape and size of particle, surface texture, hardness, mineralogical composition, abrasion, quality, porosity, etc., upon strength (particularly flexural strength) and durability of concrete.

3. Method of test for durability of aggregate.
4. Method of test to indicate directly the quality of aggregates for concrete.
5. Studies of economy in the use of different combinations of aggregates.

Concrete.

1. Method of test for workability of concrete that will be indicative of workability on concrete pavement construction.
2. Effects of admixtures.
3. Curing methods.
4. Method of test for durability of concrete.
5. Study of mixing process.

Bituminous Materials and Mixtures with Mineral Aggregates

1. The development of tests for determining suitability, proportions and characteristics of aggregates for mixed-in-place bituminous surfaces and for premixed laid cold bituminous surfaces.
2. Bituminous emulsions suitable for cold surface treatments.
3. Determination and classification of the important characteristics of bitumens for use in surface applications, mixed-in-place surfaces, etc.
4. Methods of test.

Steel.

1. Use of rail steel bars for reinforcing.
2. Use of plain vs. deformed bars, bond effect.

Miscellaneous Materials, Paints, Oil, Fence Posts, etc.

1. Characteristics of paints for traffic marking.
2. Determinative tests for lubricating oil.
3. Economic study of various preservative treatments of guard fence posts.

CONSTRUCTION

1. Effects of various degrees of surface manipulation in finishing concrete pavements.
2. Critical studies of developments in construction methods and appliances as they appear.

V. Maintenance

The problem of the maintenance engineer is that of keeping the roads as nearly as possible up to their original condition at reasonable cost. His experiments have been directed toward the improvement of maintenance methods, and the lowering of costs. Improvements in maintenance equipment have come about largely through trial on the roads, and will no doubt continue to come in this way as inventive genius does not wait on organized research projects. Keeping abreast with the improvements in methods and equipment is itself a research project.

Many economic questions depend upon the relation between amount and kind of traffic and maintenance costs as discussed under Transportation Costs. A large feature of the work of the Highway Research Board in this field is the collection and dissemination of information on improved methods and equipment as they are developed in the various states.

NEEDED RESEARCH

Maintenance

1. Maintenance of cracks and joints in concrete pavements. Characteristics of crack fillers.
2. Resurfacing of concrete, brick and asphalt pavements.
3. Effects of increasing traffic on maintenance methods and costs.
4. Data on reduction in loss of material on waterbound and traffic-bound surfaces, due to use of dust palliatives.
5. Method of testing quality of cutting edges of grader blades.
6. Specifications for quality of cutting edges of grader blades.
7. Methods for resmoothing surfaces of bituminous surfaces and pavements.
8. Methods for snow removal, drift prevention, and treatment of icy pavements.
9. Operation and care of maintenance equipment.

VI. Traffic

The sole purpose in building and maintaining highways is to provide service for the traffic and the principal highway effort is to provide such roadways that the combined costs of building, maintaining and using them safely will be as low as possible. To reach this desired end, adequate service must be provided, and vehicles must use the roads in an efficient manner. In order to design highways and formulate regulations for their use, information about the amount and behavior of the traffic is necessary. Vehicles should be able to get over the roads expeditiously, economically and safely. To these ends four factors must contribute: Improvement programs, road design, traffic regulations and education.

In order to properly evaluate each of these factors in planning regional or state improvement programs, highway transport surveys, including not only counts of present traffic but study of transportation trends and influences for estimation of future traffic, are necessary. Research work in connection with the use of highways consists largely in search for possible betterments by study of the information collected through transport surveys, traffic counts and accident reports and statistics.

Improvement Programs and Design of Roads as Affected by Traffic. The objectives are the building of highway systems that will be adequate to care for the traffic during the life of the improvements.

Traffic Regulation and Safety. The object in regulating traffic is to expedite movement of vehicles with safety. Most regulatory methods in use today have the effect of restricting rather than expediting traffic. There is a real need for research that will divorce itself from present regulations and conditions, and develop the fundamental mechanical and psychological principles upon which practice that will expedite traffic movement with safety can be based. For instance clear conception is needed as to the inherent and legal rights of each vehicle in its use of the road and in its relation to other vehicles. Also more definite ideas as to what traffic regulation should accomplish are needed. The principal problem in traffic regulation is the acquisition of basic facts upon which regulatory measures can be based so that the results will not be merely matters of opinion. The principal problem with respect to safety is how to inculcate vehicle drivers and pedestrians with safe traffic habits. The concern of the highway engineer is in so far as possible to eliminate physical hazards.

NEEDED RESEARCH

Traffic

IMPROVEMENT PROGRAMS AND DESIGN OF ROADS AS AFFECTED BY TRAFFIC

1. Standardization of traffic survey methods.
2. Methods for measurements of traffic congestion.
3. Methods for traffic classification of highways with reference to type, width and design of roadway and width of right of way.
4. Capacity and width of traffic lanes for various volumes and types of traffic.

5. Required right of way width for various roadways and most economical method of acquiring right of way.
6. Study of highway intersections as respects design.
7. Provision for parking.
8. Study of highway facilities in the vicinity of airports.
9. Study of justification for toll roads and bridges.
10. Studies of possibilities of increasing highway capacity and relieving congestion by (a) widening, (b) additional roads, (c) segregation of high and low speed traffic, (d) by-pass routes, (e) elimination of railway grade crossings, (f) elimination of highway grade crossings.
11. Basis for determination of grade crossing separation and division of cost.

REGULATION OF TRAFFIC

1. Study of rights of vehicles in relation to other vehicles and pedestrians.
2. Study of objectives in traffic control.
3. Relative efficiency and cost of methods of regulating flow of traffic through intersections, traffic police, stop and go signals, free movement.
4. Study of the importance of eliminating grade crossings.
5. Study of efficiency of speed laws.
6. Provisions for pedestrian safety.
7. Study of efficiency of signs.

SAFETY

1. Study of relation between accidents and highway hazards.
2. Study of relation between accidents and vehicle condition and characteristics.
3. Study of relation between accidents and speed of operation.
4. Study of the personal factor in highway accidents.
5. Results obtained by examination and licensing of drivers.
6. Results of different systems of highway traffic control, state police, county, local, etc.
7. Hazards of night driving. Lighting.
8. Standardization of accident reports.
9. Study of results of educational campaigns.
10. Relation between surface characteristics and skidding.

DISCUSSION
ON
REPORT OF COMMITTEE ON COORDINATION AND PROGRAM

Mr. A. J. Brosseau, *President, Mack Trucks, Inc.* I have gone over, very carefully, the outline of the program of Highway Research.

Somebody has evidently given a good deal of time to its preparation as it is a very thorough and comprehensive piece of work.

Some details appear to be omitted, although they are probably in the mind of the man who made the outline.

For instance, with respect to highway administration, where minor political units are mentioned, it is assumed that the metropolitan area is under consideration along with counties.

As long as we are getting into the question of commercial vehicle taxation in the industry, I think it would be worthwhile to inject this as a special feature in the subject of highway finance as outlined for discussion. Of course, the subject must come up under the question of highway transportation costs and it might be well to give it more emphasis.

I notice that highway administration is mentioned first. I don't know whether this is done deliberately or not, but I think that in the light of the various drives that are now being made for diffusion of funds that it becomes probably the outstanding question in highway development and one which should be given most thorough consideration.

Between the express highway plans on the one side and secondary roads on the other, the state units are being faced with a drive which might easily result in unsound diffusion of funds.

Whatever facts research can contribute to the solution of this problem should be available when discussion of it reaches those authorities who must make decisions.

There is nothing in the whole program which goes to the question of the relationships between the various types of transportation, and yet there are many questions still unanswered in this field.

The problem of traffic congestion is not stated in so many words, but I presume that it is in mind.

Mr. Alfred E. Flinn, *Director, Engineering Foundation.* In carrying on the research work of the Highway Research Board and of the experimental bureaus and the associations with which it is influential, it should be urged that all possible use be made of all published and unpublished information on highway research and experience, and from all other fields of science and engineering, even some which may not seem related. Engineering Societies Library and possibly a few others are equipped to make searches, copies, bibliographies and translations. A critical review of all worth-while literature should lead to economies in research effort and in making available for immediate use some information which is now hidden and used by few, if any, persons in active highway practise. This review of literature might well lead to one or more helpful manuals which might be sold. Of course, substantial financing would be necessary to do this work rightly.

Many of the topics suggested for research are old, some of them more than one generation. Doubtless the explanation is that some words continue to describe briefly the needs under changing conditions for more and better and more precise determinations. However, there appears to me some indication of lack of collection of knowledge gained in days gone by, of its dissemination and of its use by many who might be profiting from it. Strengthening of the Highway Board's endeavors to coordinate and stimulate efforts all over the country would probably help this situation.

APPLICATION OF AERIAL MAPPING TO HIGHWAY CONSTRUCTION

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Photography was applied to surveying long before the introduction of the airplane. Photographs were taken from points on the ground with a photo theodolite in which glass plates were placed in a vertical position. An instrument called a " Comparator " was used for measuring coordinates of points on the glass plate, after which, by the application of geometric principles, it was possible to construct a map. Terrestrial photo surveying, however, has found but very limited use. One interesting application might be mentioned—the determination of the highest peaks in the continental divide for fixing the boundary of southeastern Alaska.

Courses in Photographic Surveying have been given continuously at Syracuse University since the establishment of the College of Applied Science. On account of the limited application—and due to the broadening of the Civil Engineering Courses—Photographic Surveying was dropped by many of the American Universities.

With the advent of the airplane, a new interest has been created in Photographic Surveying, and its field of application has been greatly broadened.

In order to stimulate interest in this work, The Guggenheim Foundation for the Promotion of Aeronautics has made a generous contribution to Syracuse University for assistance in establishing courses in Aerial Surveying and Mapping.

Although German Universities and German instrument manufacturers have taken the lead in this work, there are yet many problems to be solved. It is hoped that America will contribute its share in this important field of research.

It is quite improbable that the transit will ever be entirely superseded by the aerial camera. There will always be operations demanding the use of the transit, such as establishing points on the ground for construction purposes, and the definition of property boundaries. But it is indeed probable that in extensive topographic mapping the aerial camera will in the future be used so exclusively as to reduce transit work to a minimum. Moreover, when one considers that only two-sevenths of the earth's surface has been mapped, he realizes that this work presents vast possibilities for the application of aerial surveying.

Aerial surveying has introduced a new type of map known as a "Mosaic." It is made by assembling a series of photographs—reduced as nearly as possible to the same scale—properly matched and oriented with respect to each other. Some ground control is of course required for the construction of the best mosaics.

There is in reality no possibility of having aerial photographs on a definite scale throughout, unless the exposures are made with the plates precisely horizontal, and unless the entire terrain is level. Even then, were the scale to be known, it would be necessary to know the focal length of the camera and the exact height at the instant of exposure or to scale known ground distances. Of course the ground never presents a level surface, and the other conditions are difficult to fulfill.

For instance, research has failed to disclose a method for holding the camera in an airplane so that the plate will be horizontal. Level bubbles are attached to cameras the same as they are to transits, but they are affected by the unstable condition of the airplane, and by changes in speed and direction of flight. Gyroscopic instruments have been tried to eliminate the tilt of the camera, but without success. The amount of tilt may however, be minimized to as small as one degree by a skillful pilot cooperating with an expert photographer using a suitable form of camera suspension.

Photographs are perspective projections on which the scale is the ratio of the focal length of the lens to the height of the lens above the ground. It is, therefore, apparent that even if the absolute elevation of the lens at the instant of exposure are known, areas on the same photograph having different elevations will be shown to different scales. Very sensitive barometric altimeters have been perfected, but a simultaneous reading on the ground is required for the calculation of the elevation of the lens. Experiments are being made with radio waves for determining airplane elevations. A solution of this problem would add greatly to the safety of flight not for photographic purposes but, for example, in fog when the ground cannot be seen at all.

A camera with a 12-inch cone—the commercial term for focal length—will take a photograph on a scale of 1 in 5000 at a very moderate altitude of 5000 feet above the ground. Photographs have been taken from altitudes as high as seven miles. The size of the usual photograph in American cameras is 7 by 9 inches with the 7 inches in the direction of flight. It will cover a distance of 0.55 mile on the ground. The displacement of points due to changes in elevation is greater on the edges than at the center. An overlap of 60 per cent is, therefore, required in the direction of flight; exposures have to be made every 13 seconds for a ground speed of 60 miles per hour.

Due to the required frequency of exposure as well as the additional weight of glass plates, film rolls are generally used. The Brock Camera perfected by the Brock & Weymouth Company of Philadelphia, is the only exception. Some aerial cameras have plate adapters which have a limited use. The usual film is subject to variable shrinkage in the process of development. It is claimed, however, that the Agfa Aerochrome Film, manufactured in Germany, has a uniform shrinkage. This film is now being given a field test in this country.

In spite of the fact that mosaics cannot meet the requirements of an accurate map, they are of considerable value to the engineer. These inherent defects are outlined so the engineer will know what to expect from a mosaic. They should be really called " aerial mosaics " rather than aerial maps, which will be described later.

The majority of aerial surveys executed by the numerous commercial companies in the United States are of the mosaic type. Many of these mapping companies are a part of, or closely affiliated with aviation companies. Nearly all of the Army and Navy aerial mapping is of this class. The U. S. Geological Survey and the U. S. Coast & Geodetic Survey have used aerial mapping over areas which are not accessible to land methods, such as the Mississippi River delta, and as a supplementary aid to ground methods.

Mosaics are very useful in highway planning. Frequently the planning of a highway system is delegated to a non-technical board, who are abled to interpret a mosaic equally as well as the trained engineer. Mosaics show a vast amount of detail which cannot be obtained from the usual map. Traffic congestion may be studied and improvements involving new rights of way may be determined.

Dean A. N. Johnson presented a paper last year before the Highway Research Board in which an excellent analysis was made of a traffic survey compiled from an aerial mosaic of the 29 miles from the Baltimore City line to Washington. The flight was made at an altitude of 3600 feet; 127 exposures were made in an elapsed time of 27 minutes, making exposures every 13 seconds.

There are a good many other uses for aerial mosaics and frequently they are used by several departments of city, county and state governments. They may be employed for assessment purposes, and by city and county planning boards. Many states are making special geological surveys to study materials available for highway construction. In flat country rights of way for pipe lines and transmission lines have been acquired, and the lines laid out by the exclusive use of aerial surveys.

The requirement of a 60 per cent of overlap in the direction of flight previously mentioned, makes it possible to view two photographs of the same area taken from two different positions of the airplane by means of a mirror stereoscope (Figure 2), or by a magnifying stereoscope. The ground can then be viewed in relief, and a preliminary estimate may be made of the amount of grading involved in a highway improvement. Geological formations may also be studied.

Attempts have been made to construct topographic maps from mosaics by using them as plane table sheets and determining elevations by the usual ground methods. The stereoscope is used as an aid in plotting contours. The angular value of radial lines drawn from the optical center of a horizontal photograph to any other points is not affected by the ground relief. A photogoniometer is an instrument for measuring angles on aerial photographs with a high degree of precision. By a system of intersecting radial lines, maps to a true scale may be constructed. However, more satisfactory methods for constructing topographic maps from aerial photographs which also give more accurate results are now available.

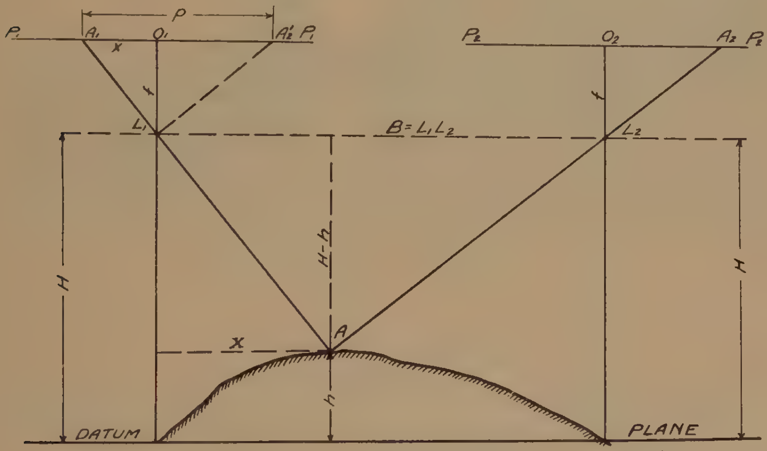
Multiple lens cameras are used to increase the width of an area covered by a single line of flight. News reports of Captain Ashley C. McKinley's Photographic Survey with Commander Byrd in his South Polar Expedition state that his photographs will cover a strip seven miles wide. This could only be accomplished at the reported altitude of flight with a multiple lens camera or by oblique views described later. The Bagley Camera used extensively by the U. S. Geological Survey and the U. S. Coast and Geodetic Survey was a three-lens camera, the two side lenses being placed at angles of 35 degrees. It has now been superseded by a four-lens camera and a five-lens camera is being planned. The oblique views must be rectified by an apparatus called a " transformer " which is a combination of a camera and a printer, before they can be used for mapping purposes.

The mounting of an aerial camera permits the taking of an oblique view as well as a nearly vertical one. An oblique view is a perspective picture which shows relief to the naked eye. Preliminary studies for bridge approaches as well as the elimination of crossings of highways by railroads at grade can be made from oblique views.

The Canadian Government has made extensive use of oblique aerial surveys in mapping their vast inaccessible areas of forest and lake country which are generally of very low relief. The distance of the horizon line from the center of the plate permits the calculation of the tilt of the camera. The focal length of the lens and the height of the camera are the only additional data necessary for compiling by the principles of perspective geometry, a plotting grid diagram by which line maps may be constructed from the oblique views. No economic method has yet been developed for plotting contours from oblique views. These Canadian maps drawn on a scale of four miles to the inch will undoubtedly serve as the basis for planning highways when sufficient resources are discovered to warrant the development of these outlying territories.

For many engineering projects including highway construction, a topographic map is indispensable. It is possible to compile topographic maps to a true scale from aerial photographs. The field work of taking the photographs is identical with that for aerial mosaics. Photographs should overlap 60 per cent in the direction of flight.

Figure 1 illustrates one of the principles used. P_1P_1 and P_2P_2 are two horizontal photographs in which the lens shown at L_1 and L_2 are at the same altitude " H " above a datum plane. The point " A " has an elevation " h " above that datum plane. The image of the point " A " is shown as " A_1" and " A_2" on the photographs P_1P_1 and P_2P_2 respectively. The focal length of the camera is " f." $L_1A'_2$ is drawn parallel to L_2A_2. The parallax of a point may be defined as the algebraic difference of the two distances in the direction of flight from the optical centers of two horizontal photographs showing the images of the same point taken from two different positions of the camera at equal altitudes above a datum plane. The parallax of the point "A" is designated by "p" and is shown by the line $A_1A'_2$ in Figure 1. Equations (1 to 7) show the method of calculating the elevation of the point "A" as well as the "X" and "Y" coordinates from the optical center of the photograph.



$$\frac{p}{f} = \frac{B}{H-h} \quad (1)$$

$$p = \frac{fB}{H-h} \quad (2)$$

$$H-h = \frac{Bf}{p} \quad (3)$$

$$\text{Parallax} = p = O_1A_1 + O_2A_2 = A_1A'_2$$

$$X = x\,\frac{H-h}{f} \quad (4)$$

$$X = x\,\frac{B}{p} \quad (5)$$

$$Y = y\,\frac{B}{p} \quad (6)$$

Figure 1. Parallax

The Stereocomparator is a foreign instrument used for measuring parallax from which the elevations of points may be calculated. The corresponding instrument in the Brock process is called a stereometer. The tilt of the camera must be eliminated before parallax measurements are taken. Correcting projectors and rectifying cameras are used. This is often the most difficult step in the process.

In the Brock method for constructing topographic maps from aerial photographs, positive glass plates are made from a set of overlapping negatives. The optical centers are marked, and the positive plates are then horizontalized and placed on a Stereometer table. (Figure 3.)

The plates are properly aligned for stereoscopic vision and the micrometer is set to read the computed parallax for points of known

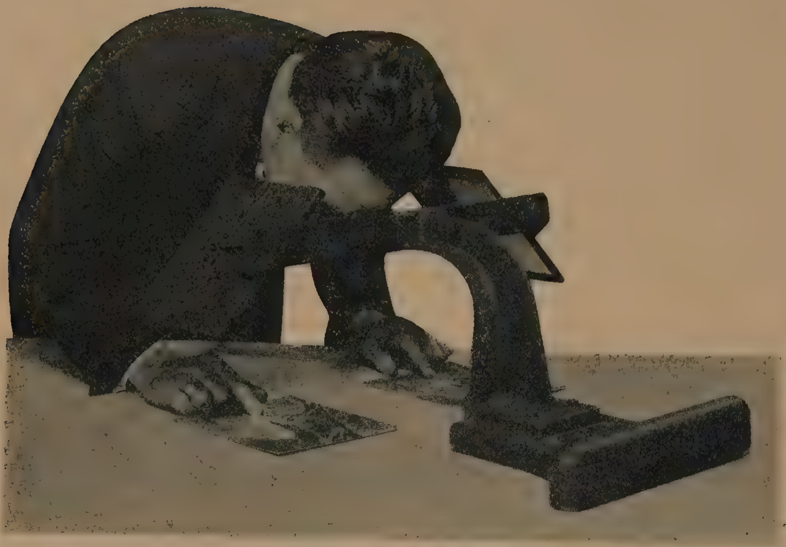


Figure 2. Fairchild Mirror Stereoscope

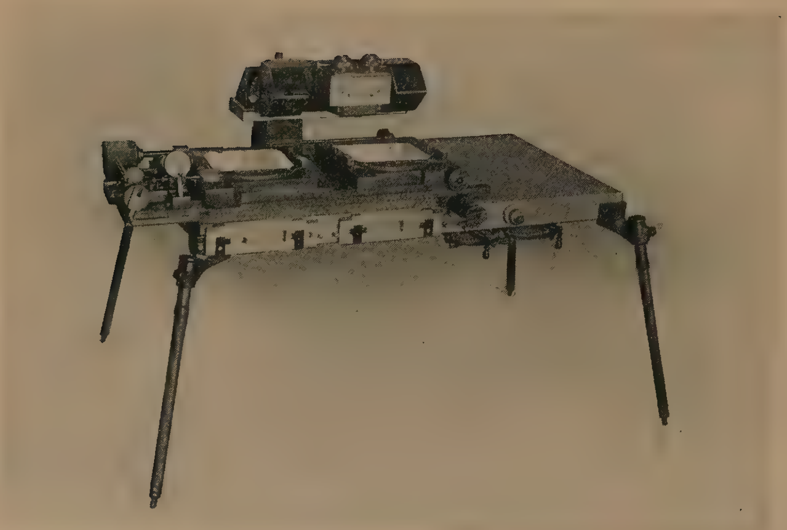


Figure 3. Brock Stereometer

elevation. The spread between the plates is then set to correspond to calculated parallax for the elevation of a contour line. A cross hair or a floating mark is made to touch the ground in the stereoscopic view and its path will make a contour line directly on one of the glass plates. As every contour line would be plotted on a different scale, it is necessary to reduce them to the same scale before combining them in a finished map.

The ground control necessary is the difference of elevation between several points on every set of plates and a known distance about every 20 miles. Great accuracy is claimed for the Brock method due to the exclusive use of glass plates instead of films, and to the extreme accuracy of the correcting projector for eliminating the tilt of the camera. An additional advantage of this process is that several sets of plates can be worked simultaneously.

German investigators have developed a process for making contour maps from aerial photographs by a somewhat different method. Stereoscopic methods are used in conjunction with a pantographic plotting device. The stereoplanigraph for the automatic plotting of maps from aerial photographs taken with axes at any angle, and developed by the Carl Zeiss Company of Jena, is now being improved. The Autocartograph, devised by Dr. R. Hugershoff about 10 years ago, has been recently replaced by an improved type known as an " Aerocartograph." (Figure 4.) Several of these machines are now in use in the United States, and are also being used in other countries.

The method of the Aerocartograph may be described as aerial triangulation, depending upon the angular values obtained from the overlapping photographs by two photogoniometers so coordinated as to furnish stereoscopic vision. It is not necessary to eliminate tilt of the photographs by a separate process; it is done in the machine itself. A set of overlapping photographs are oriented from three known points. A pointer is made to follow the ground in the stereoscopic view for an elevation corresponding to a desired contour. A pantograph can be geared to transfer the path of the contour to a drawing of any desired scale. Two drawings to two different scales may be made simultaneously.

After one pair of plates has been completed, the forward plate remains properly oriented and the backward plate is replaced by a new one. The stereoscopic view is now reversed and the new plate is

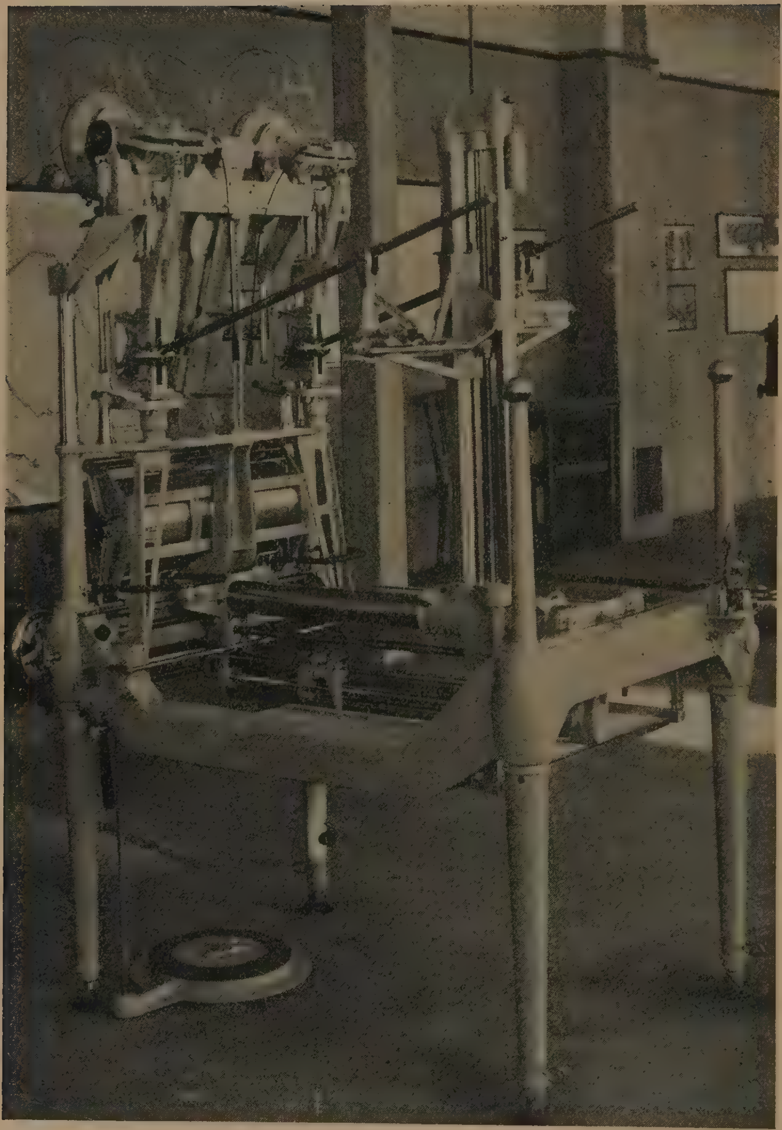


Figure 4. Aerocartograph

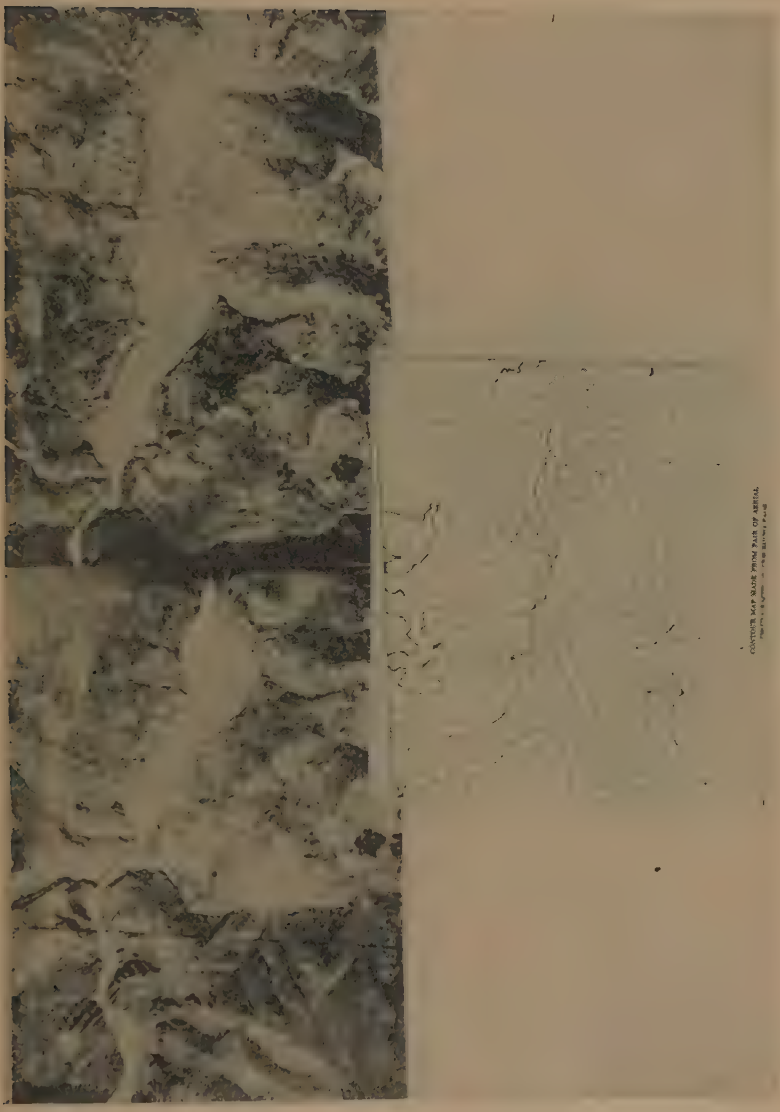

CONTOUR MAP MADE FROM PAIR OF AERIAL

Figure 5. Contour Map from Pair of Aerial Photographs

brought into proper orientation, and the process is continued. It is claimed that a minimum amount of ground points are necessary for this method as each plate is oriented stereoscopically from the preceding one, and not from a new set of points. The laborious process of rectifying the photographs is eliminated. It is also claimed that the cost of the work is much lower. Sufficient work has not as yet been done in the United States with the Aerocartograph to allow a comparison with the Brock method. Figure 5 shows the two overlapping aerial photographs and the corresponding topographic map constructed with the Aerocartograph.

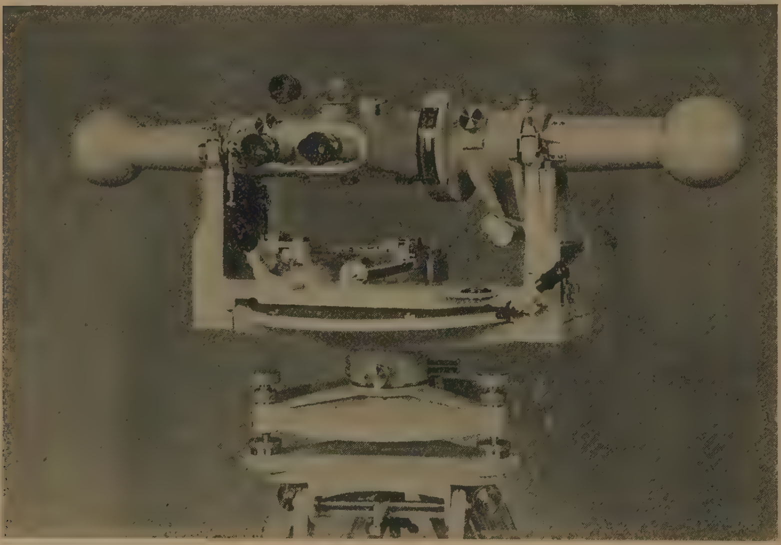


Figure 6. Stereo-Autotachygraph

Stereoscopic principles have been applied to aerial photographs in estimating the quantity of timber on a forest area. A relationship has been established between the diameter of the tree and the area of its top which can be measured stereoscopically.

The Stereo-Autotachygraph (Figure 6) is a combination of a plane table, range finder and the stereoscope. Distances and elevations of objects may be determined by merely viewing an object, dispensing entirely with the rodmen and chainmen. It has been used successfully by an American contractor in railroad construction in Persia. It may be used economically on preliminary and final cross sections in highway construction, especially in difficult topography.

The Autogiro (Figure 7) is a new type of heavier than air aircraft. It cannot be called properly an airplane because it lacks the planes or wings. It is stabilized by a revolving overhead rotor which resembles somewhat the overhead propeller of a heliocopter. It is now being manufactured in the United States. It is claimed that it will take off and land in a distance of 100 feet and can do 105 miles per hour with a payload of 400 pounds. It has the additional qualities of being able to fly low and slow at a speed of 25 miles and a few feet off the ground if desired and at 12,000 feet if necessary. It has a cruising radius of 500 miles. Such an aircraft may have possibilities of solving some of the present problems involved in aerial surveying and mapping. The expected improvements in the instruments used for Aerial Surveying may also be of great value in increasing the safety of flight.

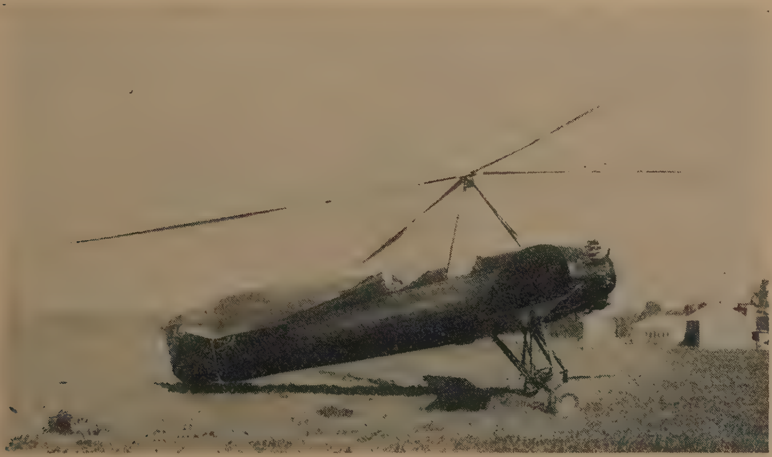


Figure 7. American Built Autogiro

SOIL INVESTIGATIONS IN RUSSIA

D. P. Krynine

Professor of Highway Engineering, Moscow Superior Technical School and Moscow Institute of Transportation Engineering

Only 15 or 20 years ago the profession of Civil Engineering had but vague ideas on the behavior of soils in foundation sites and in fills and cuts of highways and railways. Particularly among highway engineers many failures were attributed to " poor subgrade " without entering into details as to what was the real cause of the failure.

Modern engineering soil science which may be called " Applied Soil Physics and Soil Mechanics " began to develop principally in the United States about 1914, and is equally interesting to both highway and foundation engineers; the former however seem to have been pioneers in this study. In Russia too, highway engineers led in engineering soil research. The paper of Koch[1] and the consequent studies of the American soil investigators attracted the attention of Russian highway engineers about 1922, when the first years of the revolution were over, and scientists resumed their work. The author of this paper was the first in the Russian technical press to show the necessity for making use of the work of agricultural soil scientists. These men had for a long time studied soils and their properties, principally from the agricultural point of view; naturally they neglected one of the factors more important to the engineer, namely the load factor. This deficiency was to be made good.

In 1923 the present Russian Government created a special Highway Research Bureau (B. P. Gervais, chief), provided it with a laboratory and put several field parties at its disposal. Agricultural soil scientists were then called in for cooperation. The point of view from which the investigators started was as follows: Laboratory investigations alone are insufficient for obtaining satisfactory information about soil properties. A special field service must be organized for investigating and recording peculiar local conditions, such as—geomorphological data, topographic relief, and climatic conditions.

Russia has about 2 million miles of roads, the exact number being unknown. Of this enormous mileage only one per cent have a hard stone surface, and 99 per cent are common earth roads. From the very beginning Russian soil research took a different direction than that of

America, American investigations concentrating mostly on subgrade problems, while the Russians have been almost exclusively interested in earth roads, *i. e.,* in roads without hard surface. The problem of improving earth roads is one of the most important for Russia from the point of view of national economics, as is also true of all other undeveloped countries. Thus, upon the demand of the Russian delegates, the Association of Road Congresses in France included the discussion of earth road construction and maintenance in the program of the Sixth International Congress to be held in the United States in 1930.

In what follows, only four items of the activity of the Russian Highway Research Bureau will be considered: (a) preparation of soil profiles; (b) design of standard soil mixtures; (c) study of the frozen soils; (d) study of the rolling process.

SOIL PROFILES

In a soil profile soil horizons (A, B. C) are indicated, the origin of soils and their classification is specified, and levels of the ground water established; also all possible unfavorable conditions due to the presence of water must be indicated in the profile.

Since the preparation of soil profiles was entrusted to agricultural soil scientists, these profiles and maps, while made very carefully, often carry many details which do not always seem relevant to the practicing highway engineer. Yet in many cases very interesting conclusions have been drawn from such, at first glance, impractical soil profiles and maps. They, for instances, often indicate the absolute and inevitable necessity of a relocation of a road; or the impossibility of making a deep cut or ditch in a locality where the underlying soil is of a nature making it susceptible to erosion, etc.

It is hoped that a new generation of highway engineers possessing the requisite knowledge of soil science and having a broad view of Highway Engineering, will definitely standardize highway profiles and maps.

DESIGN OF SOIL MIXTURES

Further, the Research Bureau made efforts to work out standard proportions of sand and clay mixtures for earth roads' building. It was at least necessary to establish the principles on which the design of such mixtures was to be based. Of course, the Russians were aware of American attempts in the same province.[2] The basic principle of the Russian school of soil scientists is that soil is a product of both the original geological formation and the effects of local climatic, topographic and other conditions. This being so, one soon had to conclude that it was impossible to design a general standard for the whole country and that an individual one was to be sought for in each zone. However, the first attempt of the Research Bureau gave the following proportions for standard mixtures:

Grain size 0.00–0.01 mm	20 per cent
Grain size 0.01–0.05 mm	15 per cent
Grain size 0.05–0.25 mm	30 per cent
Grain size 0.25–1.00 mm	35 per cent

An attempt was made by Prof. G. A. Zaslavsky to solve the problem of standard mixture in a mathematical way based on an idea somewhat similar to that of Fuller on concrete mixes. The denser and the less porous a mixture is, the better it meets highway requirements. The voids of the coarse fractions are to be filled with finer ones, and the voids of these with clay, which is also to act as the cementing material. Samples prepared on the basis of this theoretical formula must be tested in the laboratory and possibly somewhat modified in the light of these tests.

FROZEN SOIL

Russia is known as a classical country of frost. This however, is true only for some of its regions. Near Moscow the penetration of the frost into the soil and subsoil is not more than 4 or 4½ feet. Measurements of the depth of frozen soil were made near Pskov simultaneously on the roads themselves and on the adjacent snow covered fields. The difference between the two was astonishing. In the spring after the snow melts, the frozen soil in the fields is only a few inches deep, while that under roads is three or four feet deep. This difference persists for one month or more. This means that during a certain time an enormous frozen body, something like a frozen earth dyke, is to be found under the road, disturbing drainage conditions. The study of this phenomenon is being continued.

Still more serious for Russia is the problem of permanently frozen soils which occupy an enormous area in Siberia and extend also south of the Chinese frontier. The upper layer of the permanently frozen soil thaws in summer, yet from a certain depth on, its temperature is always below freezing point. In Yakoutsk the frozen depth exceeds 350 feet, being however less in other localities (for instance, in Poustozersk 58 feet, in Taldan about 220 feet and so on).

Permanently frozen soil is as hard as rock; but under masonry structures, masonry being a fair conductor of heat, it thaws, originally hard surfaces become soft, and the structure may fail. Of course, roads constructed in such country never can be considered as stable. Even wooden bridges suffer. (Figures 1, 2, 3, 4.)

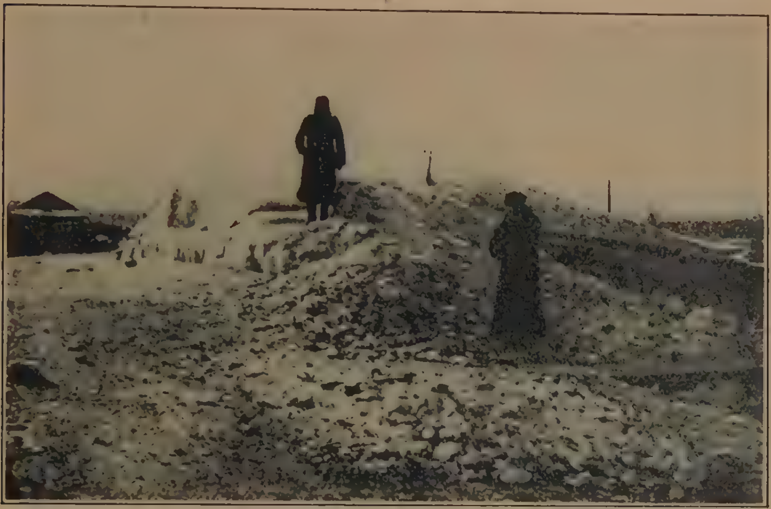


Figure 1. "Naled" or "Taryn," Successive Ice Layers Formed by Water Rising from the Ground

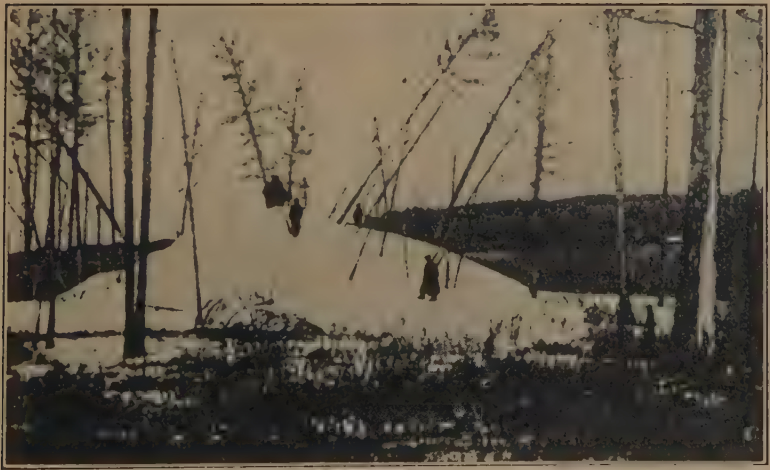


Figure 2. Frost Heaving in the Zone of Permanently Frozen Soil, 22 Feet High

A thorough study of the permanently frozen soils is necessary for Russia. Inquiries were made on similar conditions in the United

States and in Canada, but little data have been obtained. There are in Russia notable geological works referring to this state of soil,[3] but data necessary for a practicing highway engineer are not yet available.

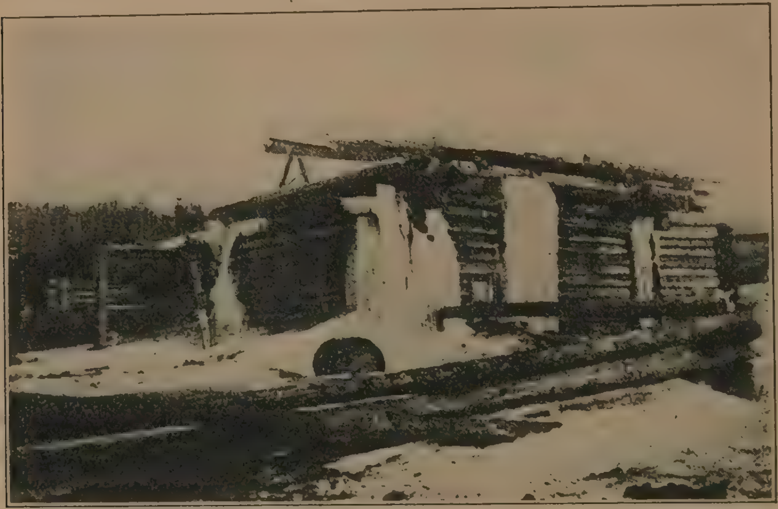


Figure 3. A Little Wooden Building Filled with Ice Formed with Water Risen from the Ground

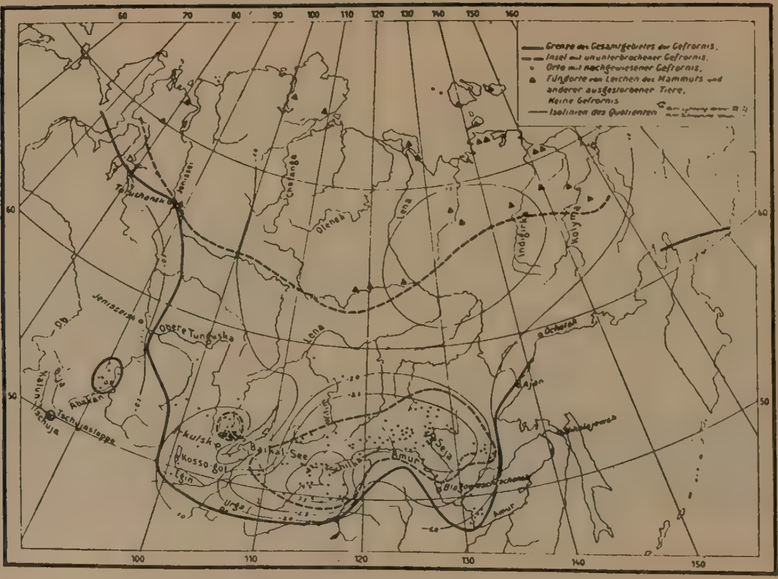

Grenze des Gesamtgebietes der Gefrornis.
Insel mit ununterbrochener Gefrornis.
Orte mit nachgewiesener Gefrornis.
Fundorte von Leichen des Mammuts und anderer ausgestorbener Tiere.
Keine Gefrornis
Isolinien des Quotienten
60
70
80
90
100
110
120
130
140
150
160
Turuchansk
Jenissei
Chatanga
Olenek
Lena
Indigirka
Kolyma
Jenisseisk
Obere Tunguska
Ochotsk
Ajan
Abakan
Baikal-See
Kosso-gol
Urga
Amur
Seja
50

Figure 4. Distribution of Permanently Frozen Soil (Occupies Approximately Half the Area of Europe)

At the present time when many American engineers go to Russia as consultants, I think that it is worth while to tell you of the peculiarities of that country. It must be also recalled here that the prevailing traffic in Russia is not automotive but horse-drawn which of course has its specific effects on the road.

ROLLING OF EARTH ROADS

Finally let us look at results of Russian experimental studies on rolling of earth roads. It has been demonstrated that during the rolling process not only soil compression takes place, but also a horizontal displacement of the soil particles both on the top and within the interior mass of rolled fill. The amount of this displacement which is often considerable, depends on the value of the coefficient of internal friction of the soil. The denest layer of rolled soil is to be found not on the top but somewhat below. Also, a certain water content is necessary to make the rolling effective.

RESEARCH

I wish now to tell you briefly of some research problems studied in the laboratory under my direction—the Highway Engineering Laboratory of the Moscow Institute of Technology, which when literally translated is called " Moscow Superior Technical School." Our laboratory in Moscow occupies approximately the same modest area that is occupied by the soil laboratory of the Massachusetts Institute of Technology, and has about the same equipment, with the exception, naturally, of the outdoor large scale experimental retaining wall test.[4]

Many efforts have been made by the author's laboratory staff to bring the mechanical analysis of soil to a certain degree of perfection. A special device called " Opaloscope "[5] has been designed and constructed by the brilliant young physicist Mr. G. I. Pokrowski (of the atom of time fame and author of a great number of papers on optics) in order to study optically the size and shape of soil particles in suspension. In these experiments it was also demonstrated that the well known Stokes' law is not applicable to particles below 0.3 μ (0.3 micron).

A synthetical study is now being made to determine what degree of accuracy in mechanical analysis is really needed in highway engineering. Mechanical analysis is a troublesome operation, and time and work saved on it are of great importance.

We use Atterberg's tests and also other routine tests, as water content determination, saturation capacity etc.—practically in the same way as it is done in this country. " Standard Compression Test " recommended by Dr. Terzaghi also will be soon adopted in our Moscow laboratory. We may seem late in this, but I recall that also in the Arlington laboratory of the U. S. Bureau of Public Roads this test is not very extensively used for highway purposes.

We work on some problems in soil physics and soil laboratory practice. Amongst them are: (a) scale like nature of clay particles; (b) entrapped or pinched air in soils; (c) optical methods of determining water content of soils without disturbing the regular march of the experiment as one is obliged to do when the usual procedure is followed; (d) water movement in soils. I will say a few words on each of these problems.

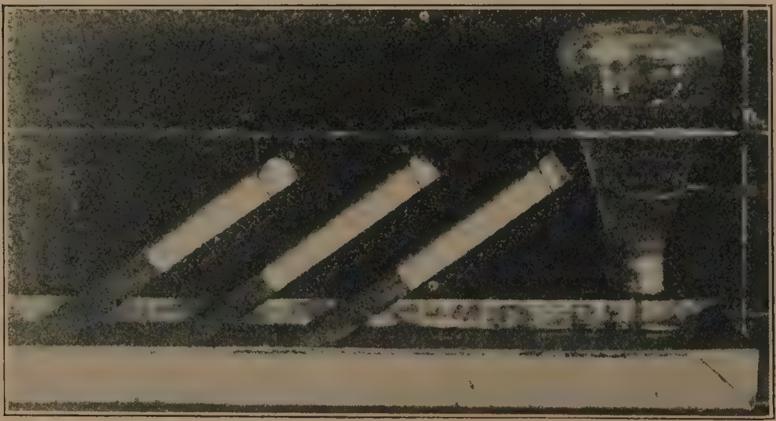


Figure 5. Capillary Movement of Water in Inclined Tubes Filled with Clay Powder

On the scale-like nature of clay particles, I have already published an article in " Public Roads."[6] The method merely consists in filling a glass tube with dry clay as it is done for studying the capillary movement and observing the tube immersed in the water not vertically, but in an inclined position. Observations show that the movement of the capillary water is not vertical, but parallel to the inclined axis of the tube which, of course, suggests a laminary structure of the clay. (Figure 5.)

In the actual state of our knowledge of soil mechanics, no one doubts that wet clay represents a system of soil and water intimately bound together. Besides this, bubbles of entrapped air, often microscopic in size, may be dispersed in the water and pressed against the soil particles with a pressure intensity which is determined by the state of stress in the water.

A glass tube about 1 inch in diameter was filled with dry or rather hygroscopically moist clay powder to the height of 4 inches. On the bottom of the tube a cheesecloth screen was tied to prevent the soil from falling out; instead of the cheesecloth a perforated metallic bottom may be used. The tube was immersed in 4 inches of water; then for 35 days from time to time the tube was taken out of the water, carefully dried with a rag and weighed. The barometric pressure and temperature were recorded at each weighing. It was also found that the weight fluctuated owing either to contraction or expansion of the bubbles of entrapped air in accordance with the laws of Boyle-Mariotte and Gay-Lussac. As the air contracts, the water enters into the soil in the tube, and its weight increases. On the contrary, when the air expands, the water is driven out of the tube, and the total weight decreases.

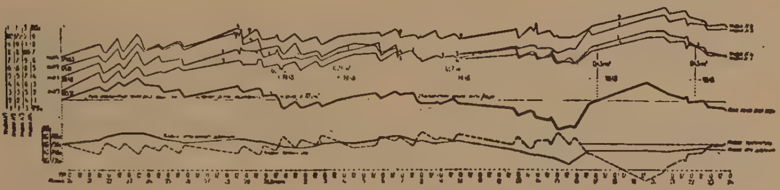


Figure 6. Fluctuations of Weight of Tubes Filled with Saturated Clay Due to the Presence of Entrapped Air

There is an exception to this rule which corresponds to a violent decrease in atmospheric pressure. In this case the air seems not only to drive the water back, but forces its way through the water and escapes. Therefore an increase of weight instead of a decrease is to be noted in this case. (Figure 6.)

The author of this paper cannot say whether considerable quantities of air may be entrapped in the deep layers of ancient natural clays. But as to relatively modern deposits, especially artificial fills, there is no doubt of the presence of entrapped air therein.

Thus water seems to penetrate into the interior of artificial fills during a decrease in barometric pressure or a considerable rise of temperature. The combination of these two factors may be dangerous to newly constructed railroad or highway fills during floods or heavy rain storms. It is also true for poorly constructed earth dams as well. Any fresh fill will probably show considerable settlement during a continued decrease of barometric pressure following a rain. The practice of dragging graded earth roads in damp weather seems to be justified because it tends to drive the entrapped air out and thus consolidate the central portion of the fill.

In laboratory work one must carefully drive out all the air from soil samples to be tested, in order to obtain accurate results.

Now I pass to the determination of water content of a soil. Under actual conditions of laboratory practice this operation is exceedingly simple in itself. But it involves the extraction of samples from the soil tested, which practically must be considered as causing serious interference or even destruction of the experiment, especially if water movement in soils is studied. In order to avoid troubles of this kind optical methods have been devised. The previously mentioned Mr. Pokrowski designed and constructed an apparatus called "Reflectometer"[7] which permits the determination of the water content by observation of the sample through the walls of a glass tube or glass box, without touching. The water content modifies the color of the soil and its reflecting capacity. If the investigator is in a position to measure the reflecting capacity and if he knows the relation between this capacity and the water content of a given soil, he can determine the water content very easily. For handling the reflectometer, no special knowledge of physics is required.

The reflectometer together with other apparatus was applied to the study of the laws of water movement in soils, especially those of capillary movement and of percolation of rain water into the ground. Some conclusions have already been drawn and published in part,[8] but the bulk of the work is yet to be done.

Such was the state of soil investigations when the author left Russia at the end of July, 1929.

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3. Writings of Schestakovitch, Soumgine, Nikiforoff and others.
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DISCUSSION
ON
SOIL INVESTIGATIONS IN RUSSIA

Mr. I. B Mullis, *U. S. Bureau of Public Roads.* The speaker has read this splendid paper by Prof. Krynine with great interest. Russian scientists have already contributed much to the science of agricultural soils and it is confidently believed that they will perform their latest mission equally as well in furthering the science of road construction.

The major activity of the Russian Highway Research Bureau seems to be directed to the solution of the problems of earth road construction and maintenance. This is a problem which confronts not only Russia but the entire world. In the United States of America we have about 3 million miles of roads, of which about 626,000 miles are surfaced, leaving more than 2¼ million miles of earth roads in a more or less unimproved condition.

Regardless of the extent of transportation developments, we must continue to provide earth roads where the limitations of traffic and finance make the higher types impracticable. But to make a system of roads most efficient, each type must be developed to its highest degree of economic efficiency. This can be accomplished only through research.

In the days when macadam surfaces constituted the major type, it was universally recognized by highway engineers that hardness, toughness and density of the surface mass were highly essential characteristics. In the development of gravel, sand-clay and topsoil surfaces, similar characteristics have been generally sought.

In Dr. Strahan's classical work [1] on semi-gravel, topsoil, sand clay and other road materials in Georgia, he stresses the desirability of the hardness of sand and gravel for furnishing the essential resistance to wear; a graded mixture of coarse and fine inert particles for producing the essential mechanical bond and increasing the density; a suitble quantity of good strong cementing clay for binding the mass and resisting the penetration of water; the manipulation of the mixture in a wetted state for the purpose of puddling and spreading the clay over the particles of sand and silt; and lastly maximum compaction by various means equivalent to tamping by blunt pointed implements such as horses hoofs, narrow tired wagons, multiple ring rollers, etc.

Strahan also specified certain minerals and materials which on account of their softness or other undesirable characteristics are not suitable for use in road surfaces. Those listed by him are: feldspars (presumably the weathered ones), micas, schists, hard pan or other soft friable material.

Recently Hogentogler and Terzaghi[2] have, in a very comprehensive paper grouped uniform subgrades, arranged according to the characteristics conspicuous in their performance. They have divided these subgrades into 8 groups ranging from highly stable topsoils, and less stable clays to the least stable materials such as mucks, micaceous and other soft or elastic soils in various states of moisture. In this grouping essential characteristics, such as cohesion, friction, etc., have received careful attention.

In the time allotted only a brief summary of some of the more important features of certain common rocks, minerals and soils and their suitability for fill or surface materials seems possible.

According to Dr. Lord[3] the resistance to wear of igneous and metamorphic rocks containing an abundance of hard minerals such as quartz is greater than that of similar rocks rich in softer minerals such as mica, serpentine, calcite, etc. Foliated metamorphic rocks are as a rule deficient in toughness and are therefore not well adapted to road construction. Sedimentary rocks are usually deficient in wearing properties, except in the case of highly indurated sandstones, containing a moderate amount of siliceous clay cement and limestones or dolomites rich in quartz and having very little clay.

Tests made by Kellerman[4] show that the percentages of wear and absorption in crushed stone, slag and gravel increase in a general way with decreasing densities.

Pure quartz sand is extremely hard, usually consisting of irregular rounded particles without cleavage and forms one of the most durable and useful materials used in road construction. As a material for fills and road surfaces it is most useful when associated with very limited quantities of clay which act as a water resistant and also as a binder. This added clay also reduces the permeability and increases the density of the mass—two additional characteristics which seem important.

Sands formed from frothy or vesicular lava may be quite hard due to the presence of glassy or other hard substances but possess little toughness, are light in weight and usually possess a high capacity for water. Only the denser varieties of these sands have much, if any, value as a material for roads.

Pure quartz silt may probably be found in very limited areas, but it is most commonly found mixed with sands and clays. As a rule it exists as particles of rock minerals or rock flour in varying states of weathering ranging from that of fresh rock particles of more or less hardness to that of material which may be as soft as talc. The particles may be of almost any shape and the pore space may vary from that possessed by sand to enormous percentages such as that of Celite or other microscopic aggregates all of which are extremely light in weight and are not adapted to earth road building purposes.

For convenience clays will be grouped into three classes, viz., ball or highly plastic clays, kaolins and intermediate types. Ball clays are characterized by their great density strength and toughness when puddled and thoroughly dried, their great plasticity when wetted and their high shrinkage upon drying. In the thoroughly compacted and cemented state Bleininger and Howat[5] found that certain ball clays possess a tensile strength ranging from 135 to 210 pounds per square inch. Kaolins on the other hand while generally plastic require less water for producing workability, are less dense upon drying from the puddled state and possess less strength than the highly plastic clays. The tensile strength according to the tests by Bleininger and Howat ranged from 104-147 pounds per square inch. According to Segar[6] the cementing substance contained in kaolin, dries to a soft powder-like mass of low strength while that contained in ball or highly plastic clay dries to a horn-like mass of considerable strength. The intermediate clays of course possess characteristics between the types just described.

Chalk and other light-weight and soft minerals absorb large quantities of water, erode easily and are not desirable road materials.

Sands, silts or clays containing vegetable matter either in the form of humic solutions or in states of more or less decomposition absorb water in greater abundance and tend toward higher porosities than similar materials free from humus.

From information obtained chiefly from ceramists it seems that ball or highly plastic clays should furnish a denser and tougher bond for sand-clay and gravel mixtures than that furnished by kaolins possessing low cohesive properties. The major difficulty, however, in making this suggestion practical seems to be that of uniform mixing such clays with sands. It seems that fine grinding or the reduction of the clay to a slurry of buttermilk-like consistency before mixing with the sand are possible methods.

Regardless of the type of material, maximum compaction seems always essential. To accomplish this most effectively, the moisture content of the material to be compacted and the method of compacting are of paramount importance. Sand-clay mixtures for road surfaces should first be puddled [1, 7] and then be compacted as rapidly as drying permits by well distributed traffic or by a tamping roller.[1]

In compacting highly plastic clays a suitable moisture content is essential, otherwise the rolling will be less effective. When too dry, the soil will be incoherent but if too wet it will be " spongy " and therefore cannot be compacted. The most suitable moisture content for rolling is when the soil coheres upon being squeezed in the hand but not wet enough to " ball " or show plasticity; this is the moisture content equal to that termed by farmers as the " optimum moisture condition " for cultivating crops.

For compacting earthwork the tamping or multiple ring rollers seem to be more effective than the usual type. On one earth dam in California [8] where sprinkling to the moisture state just suggested was practiced, followed by the use of sheep's foot tampers, the 85-foot dam settled not more than 0.1 foot in 2 years. A similar method of sprinkling followed by the use of a 10 to 12 ton 3-wheel roller has also been found to be an essential factor in reducing soil shrinkage in adobe subgrades in Stanislaus County, California.[9]

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Mr. C. A. Hogentogler, *U. S. Bureau of Public Roads.* Professor Krynine's very interesting and valuable discussion of Russia's investigations discloses the fact that in common with other investigators, Russia is seeking an answer to the all important questions:

1. How can soils be best identified in the laboratory, and
2. What is the significance of the laboratory identification of soils with respect to their performance in service?

These questions are equally important to the highway engineer, the foundation engineer and the agricultural engineer, and they must be answered before permanent progress can be made in the sciences based upon soil performance.

Of special importance to the highway engineer is the third question: What is the practical significance of the performance of soils in service with respect to road construction?

Progress in appreciable amount has been made by the U. S. Bureau of Public Roads in its efforts to answer these three questions.

The investigational procedure which furnished this progress consisted of the following:

1. Surveys of roads in service by both engineer and soil scientist to determine both the soils or soil profiles on which failures occur and the character of the failure.
2. Laboratory investigations to determine the soil constituents responsible for particular subgrade performance.
3. Laboratory investigations to determine the soil tests best suited to disclose the presence of those soil constituents which exert a dominating influence upon the serviceability rendered by road surfaces.

For assistance rendered in this connection, special credit is due Dr. Charles Terzaghi, who furnished the principles of soil physics which explain the significance of the laboratory tests and W. I. Watkins, the soil scientist, whose keen appreciation of the importance of relatively small details in the mapping of soil profiles facilitated the correlation of the results obtained in the laboratory with those furnished by the field investigations.

The results furnished by these investigations disclose that the degree of such physical characteristics as internal friction, cohesion, capillarity, shrinkage, expansion, elasticity and ability to heave during frost action possessed by soils is dependent to a large extent upon the presence of such soil constituents as sand, silt, clay, colloids, diatoms, mica flakes and organic matter. Furthermore, the presence of these important constituents is disclosed with a fair degree of precision by relatively simple test constants as follows:

1. The lower liquid limit or the minimum moisture content possessed by the soil when in the liquid state.
2. The plasticity index or the moisture content of the soil when at the yield point subtracted from the minimum moisture content when in the liquid state.

3. The shrinkage limit or the moisture content of the soil when compressed to minimum volume due to capillary pressure.
4. Centrifuge moisture equivalent or the moisture content of a soil when compressed by a centrifugal force equal to 1000 × gravity.
5. Field moisture equivalent or the moisture content possessed by a soil when refusing to absorb additional water dropped upon the surface of the soil cake.

For obvious reasons, the mechanical analysis has but minor significance with respect to the identification of fine grained soils.

Soil identification by means of the test constants depends upon two facts:

(a) The results furnished by tests performed upon a material possessing particular physical characteristics bear a definite relation to each other and

(b) The interrelationship existing between test results changes as physical characteristics possessed by the tested materials change. In order to construct a practical soil identification chart, therefore, it was only necessary to determine by means of laboratory tests, the individual interrelationships existing between the test results possessed by the soil constituents found by the field surveys to exert an important influence upon the serviceability of road surfaces.

I wish to emphasize that the little success which we claim to have obtained in the development of a rational method of identifying soils is due mainly to the fact that we studied the properties possessed by the individual soil constituents separately, in combination with each other, and in combination with natural soils. Only by this method of attack were we enabled to understand the properties possessed by natural soils. After the characteristics of the important soil constituents were determined, the presence of the various constituents in natural soils was easily recognized.

Along with this method of soil identification there was developed also a tentative grouping of subgrades.

In this grouping, sand instead of being a material which possesses merely a given grain size becames group A-3 material which possesses internal friction, but neither cohesion nor detrimental capillarity. Members of the sand group are identified with respect to the ability possessed by their grains to resist sliding over each other.

Silt, instead of being a material which possesses only a given grain size is divided into two groups: A-4 subgrade which possesses internal friction, capillarity in appreciable amount and neither cohesion nor elasticity in appreciable amount; and A-5 subgrade which possesses internal friction, both capillarity and elasticity in appreciable amount and no cohesion.

Likewise, the clays are divided into two groups A-6 subgrade which possesses both cohesion and capillarity in appreciable amount, but neither internal friction nor elasticity; and A-7 subgrade which possesses cohesion, capillarity and elasticity in appreciable amount but no internal friction.

Thus, for two soils possessing equal grain size, one possessing a liquid limit equal to 40, a plasticity index equal to 14, and a shrinkage limit equal to 20, would be a group A-4 subgrade and the other possessing a lower liquid limit equal also to 40, a plasticity index equal to 10, a shrinkage limit equal to 34, would be an A-5 subgrade.

The soil designated as the A-4 subgrade should be thoroughly consolidated by means of a heavy roller and may be covered with a macadam if desired. This soil will function well in fills and will be improved for use in hydraulic fills by additions of sand. Important frost heave is not to be expected in this soil when occurring in the dense state and when the ground water elevation is moderately low.

The soil designated as A-5 subgrade in contrast, is not apt to be permanently consolidated by rolling. Without treatment it is not apt to function satisfactorily as subgrade for macadam. It is not suitable for use in hydraulic fills, is difficult to drain, and is apt to suffer important frost heave unless the ground water elevation is exceptionally low.

This method of soil identification under discussion is disclosing more and more that the performance of soils under service conditions is to a large extent dependent upon the presence of important soil constituents.

Consider for instance dense clay. In the field its capillary rise may be unimportant, it would form hard clods, would not drain well and consequently may not be productive of good crops. It may serve well in a core wall in a dam. As subgrade it is apt to be impermeable, may be consolidated, and is not apt to suffer important frost heave. And according to laboratory test it would be an A-6 subgrade.

Merely adding lime to this soil changes its character very appreciably. In the field its porosity is increased, its capillary is increased, it drains better and consequently is apt to be productive of better crops. Its increase in permeability decreases its efficiency as core wall material. As subgrade it is apt to suffer detrimental volume change and its possibilities for suffering detrimental frost heave are increased. According to laboratory test it becomes a group A-7 subgrade.

By the further addition of soda ash, both the soils field and laboratory performance is apt to change back to that indicated for group A-6.

The mechanical analysis is not apt to disclose the difference existing between these two soils. Therefore, according to mechanical analysis we have but one soil performing in two different ways in service. According to the results furnished by the physical tests we have two different soils each performing in its particular characteristic manner in service.

Consequently in the final mapping of our soil profiles, the different soils comprising the profiles are designated with respect to their efficiency as subgrades as A-3, A-5, etc., and not with respect to their identification as used for agricultural purposes.

In conclusion it is to be emphasized that the results of our subgrade investigations in their present status furnish a rational basis which might well be used in soil research performed for any purpose. In reaching this stage of progress we feel that we have really reached the beginning instead of the ending of an investigation of road design with respect to subgrade support.

Prof. F. H. Eno, *Ohio State University*. I feel that it is a happy event that we may have Professor Krynine with us at this ninth annual session of the Highway Research Board to bring us a brief paper on Russian soil study.

It is a satisfaction to know that Professor Krynine and his coworkers have arrived at many conclusions paralleling the conclusions reached in the United States; namely, the *use* of the work done by Agricultural soil scientists, the need of carrying the research into practical field experiments, the need of soil-minded Highway engineers, the need of standardized highway profile and topographic maps showing soil areas and profiles as well as surface topography, and the arriving at an approximate soil mixture that would make a fairly stable earth road.

The United States like Russia has more than 2,000,000 miles of earth roads, many thousands of miles of which should be cheaply treated in some way so that the farmer may be able to reach the state and county systems. Probably the cost should be kept below $2000 per mile, outside of the cost of grading and drainage structures.

The experience in Ohio seems to show that wherever it is necessary to cut through the A horizon into the B or C horizon there is a distinct danger from trouble in the sub-grade of the road. The greater porosity of the top soil and the increasing tightness of the soil horizons below slows up percolation, causes saturation along the plane of the reduced porosity and this saturation forms seepage lines, springs, and mud pockets.

The Russian attempts at soil mixtures for stabilized surfaces are checked fairly well by the conclusions reached by Professor Strahan of Georgia. His results are approximately as follows:

Clay-silt	.0 to .02 mm. in diameter	12 to 18 per cent
Silt-sand	.02 to .074 mm. in diameter	5 to 15 per cent
Sand	.074 to .221 mm. in diameter	20 per cent
Sand	.221 to 1.40 mm. in diameter	45 to 60 per cent

Professor Krynine's statements regarding the problem of handling bridge foundations upon permanently frozen soils brings up the thought whether it would not be possible to core drill the frozen soil, set in piles to such depth that the heat would not thaw the supporting soil around the bottom of the pile and by this means get a permanent undisplaceable foundation for both wooden and masonry bridges in that region.

I agree with Professor Krynine upon the desirability of determining to what degree of fineness it is necessary to go in making a mechanical analysis of the soil. It has seemed to me that if the analysis has been carried to the point where the volume of soil below .003 or .002 m. m. in diameter has been determined with the sub-divisions of soil above that point, sufficient information will be given regarding the granular composition of the soil.

Professor Krynine's conclusions regarding the character or shape of soil grains as he interprets the capillary action shown in the illustrations of his article in the Public Roads for January, 1928 may be questioned.

In the case of sand, the capillary water surface in the tube of sand rises parallel to the surface of the water in the pan due to the large pore spaces in the sand and the absence of friction to flow in such spaces, as Professor Krynine says.

In the case of the fine grained soil, however, it is my opinion that the fact that the surface of the capillary water remains nearly normal to the axis of the tube *is not due* to the form of the soil particle so much as it is due to the slowly moving capillary action held in restraint by the friction to flow caused by the exceedingly small capillaries. This slow movement permits capillary water to advance as far on one edge of the tube as upon the other edge for, both edges are placed in the water at the same instant and the action is similar all the way across the tube.

If the illustrations are examined closely it will be noted that there is a slight tilt to this capillary surface which may be accounted for by the slight difference in head of free water on the end of the tube.

In order to check Professor Krynine's statement similar tests were made with Ottawa quartz sand which is quite generally known by American engineers. It is a pure quartz natural sand as nearly spherical grained as any sand could be. This was sifted into the usual size by a set of sieves and each size tried in tubes as Professor Krynine illustrated in his January, 1928, article.

The sizes acted as follows:

That passing a 35 and retained on a 48 sieve rose parallel to the source of supply.

That passing a 48 and retained on a 65 sieve rose nearly at 90° to axis of tube.

That passing a 65 and retained on a 100 sieve rose practically at 90° to axis of tube.

That passing a 100 and retained on a 150 sieve rose practically at 90° to axis of tube.

That passing a 150 sieve rose practically at 90° to axis of tube.

The same action occurs with these spherical grains of sand as that observed by Professor Krynine.

In the illustrations in Professor Krynine's article published in Public Roads of August, 1929, are two illustrations of an ellipsoid of soil formed by capillary action, which seem to prove his conclusions, but one of these balls is almost spherical and appears elongated on the bottom, caused undoubtedly by the added force of gravity upon the capillary moving water. In figure 3 of this article, however, the ball *is* elongated horizontally in a small amount, but if Professor Krynine's argument is good the ball should be more markedly flattened.

Minerals in breaking down into fractions follow the lines of least resistance which are marked out by the form of the crystallization. It stands to reason, therefore, that the general form of the particle will be the same for the fine fractions that it is for the coarser fractions. This statement seems to be true of sizes observed down to those requiring 200 magnification.

I believe, therefore, that the angle which the surface of capillary water makes with the axis of the soil tube is dependent largely upon the size of the capillary modified by the smoothness or roughness of the particle surface.

One practical application of Professor Krynine's discussion of entrapped air occurs to me. It is connected with the compression of the soil due to rolling. This compression reduces the pore space in soils, therefore, it increases the percentage of that pore space which is filled with water at the time of rolling. This in turn reduces the extent to which capillary water from below can rise into the soil beneath the pavement because free air is entrapped and cannot give way to the capillary water beneath.

Such action is beneficial to better roads *because* the soil left in this condition beneath a paved surface will remain longer, if not permanently, in more stable condition preserved from saturation by this inability to displace the air with capillary water from beneath.

Dean Anson Marston, *Iowa State College.* I will say a few words about Professor Krynine's paper which I was asked to discuss. It was a very interesting paper and the subject is important. It is important not only to highway engineers but also to engineers in general. The science of foundations is just really beginning and we will never be satisfied in our engineering work until we are able to give some rational explanation of the various foundation phenomena which we observe. I wish to emphasize the idea which Dr. Krynine has advanced, that this study is not one that can be carried out adequately in one or merely two or three localities. It requires investigations widely distributed geographically. Localities require individual study in determining their particular characteristics. I am glad that in the United States fine work is being done in this line by the Bureau of Public Roads. I was much interested in October to listen to a paper at the meeting of the American Society of Civil Engineers by Dr. Terzaghi's successor, a member of the staff of the Massachusetts Institute of Technology. He told about the fine work being done there and which is being carried on, although Dr. Terzaghi unfortunately has gone back to Europe.

At Iowa State College we have done considerable research in these lines. I was assured just before I left the state that a bulletin giving the results will be ready very shortly. It is a cooperative bulletin of the Bureau of Public Roads and the college. It will contain a very interesting confirmation by our Professor Griffith of the correctness of using Boussinesq's formulæ in determining the pressures downward through soils from concentrated loads at the surface. Professor Griffith has verified the formulæ by measuring the displacements in the soil at different depths. He finds that they correspond closely to those calculated by the Boussinesq formulæ. Regarding frozen soils we too have had much experience with them in Iowa, although they do not remain frozen. They wreck our roads for about a month every spring.

REPORT OF COMMITTEE ON HIGHWAY TRAFFIC ANALYSIS

George E. Hamlin, *Chairman*
Superintendent of Maintenance, Connecticut State Highway Department

TOLL BRIDGES AND TOLL HIGHWAYS

Publicly and privately owned toll bridges and privately owned toll highways have been constructed in the United States to provide traffic facilities not furnished by free bridges and highways built by cities, counties and states. Both toll bridges and toll highways have been built because public funds have not been considered available for their construction at the time of their conception. While these are the usual reasons for permitting the construction of toll bridges, toll highways may be discussed from other viewpoints.

The construction of a toll highway also may be considered from the standpoint of providing traffic facilities of an exceptional character, in addition to and supplementing facilities furnished by a public highway or highways. A toll highway of this type might be constructed on a private right-of-way with light grades, long curves, without railroad or highway grade crossings, and having a roadway width for four or more moving traffic lanes. Under proper regulation, it is conceivable that on such a highway it would be permissible to travel safely at higher speeds than is ordinarily allowed on state or county systems. This feature of operation is characteristic of the famous toll highways of Italy which are built as described above.

From the viewpoints of providing additional traffic facilities and the possible reduction of traffic congestion on specific bridges and sections of highways upon which no toll charges are levied, your committee is of the opinion that the dual subject of toll bridges and highways comes within the scope of its activities and therefore it has placed this subject upon its research program for the coming year.

TRAFFIC PLANNING IS AN ENGINEERING PROBLEM

Present-day municipal traffic planning is primarily an engineering problem. For the solution or maximum amelioration of the complicated and ever-growing traffic problems of cities, scientific methods must be used. Comprehensive fact-finding, and thorough fact-analysis of the entire problem must be bases for improvement plans, instead of unproved opinions and recommendations based upon an incomplete or incorrect conception of the problem as a whole.

The engineer is trained in scientific approach to problems; hence, traffic planning work should be under the direct charge of a traffic engineer.

It should be recognized that the qualifications, training and experience of the police do not fit them to direct traffic engineering work. On the other hand, especially trained traffic officers must retain the important traffic functions of manual traffic dispatching, street training and education of drivers and pedestrians, and the exceedingly important functions of supervising and enforcing traffic law observance.

As between traffic engineering work and police traffic work, there should be no conflict, provided that each organization limits itself to its own work. There should be a close cooperation between the two divisions. Enforcement of the law is vitally necessary to the success of the efforts of the traffic engineering bureau. Without such cooperation in the enforcement of the law, the best plans of the traffic engineer and the most judicious enactments of the legislative body will be of no avail.

MOTOR VEHICLE PARKING AND STOPPING

In connection with investigations relative to motor vehicle parking and stopping, the committee has adopted the definition of parking included in the "Modern Municipal Traffic Ordinance" drafted in 1928 by a Committee of the National Conference on Street and Highway Safety. The definition is as follows: "The standing of a vehicle, whether occupied or not, upon a roadway, otherwise than temporarily for the purpose of and while actually engaged in loading or unloading, or in obedience of traffic regulations or traffic signs or signals."

Parking is a desirable privilege. However, there are certain conditions which warrant the withdrawal of the privilege. Thorough study should be the basis of recommendations on this important subject.

Parking at curbs reduces the traffic capacity of a roadway. In recognition of this fact, many municipalities have prohibited parking at one or both curbs on certain streets for periods when traffic is especially congested. In some cases no parking at any time is allowed at one or both curbs, as, for example, on heavily used business streets and on some through or arterial streets whose roadways are only of sufficient width to provide the number of traffic lanes required for moving vehicles.

In many instances, especially in business districts, the prohibition of parking does not render available the full traffic capacity of a roadway because of the stopping of vehicles at the curb " temporarily for the purpose of and while actually engaged in loading or unloading." It is a fallacy, however, to presuppose that the traffic capacity of a roadway is not increased by the prohibition of parking when stopping temporarily is permitted. It has been observed that in many cases, especially during peak traffic hours, a traffic lane at the curb has been free of standing vehicles for the full length of a block. Furthermore, double parking, and loading and unloading from the second lane, are practically eliminated, a factor of prime importance in many cities from the standpoint of reducing traffic congestion.

The Pittsburgh 1927 Traffic Survey showed that based on returns from patrons and employees of 48 stores, the distribution as to modes of transportation used in entering the central business district was as follows:

	Per cent
Street cars	63.6
Automobiles	13.3
Trains	12.4
Walked	7.0
Motor coach	3.7
	100.0

This Survey also showed that only 2.8 per cent of the patrons of 48 typical stores in the central business district parked their automobiles at the curb.

It has been developed by a recent Philadelphia survey to determine the relationship between parking and shopping that out of every 100 customers in the downtown stores 8 came by automobile parked at the curb, 69 by public conveyance, 8 by automobiles parked in garages and 15 on foot.

A report of the Committee on Street Traffic Economics of the American Electric Railway Association gives the following statistics: " In Chicago in 1926 before complete ' no parking ' rules were established those parking at the curbs during a business day were only 3 per cent of all persons who entered the district. They were but 17 per cent of all who drove into the district by automobile, and of

96,082 customers at business establishments who were interviewed only 1.57 per cent were those who used parking space at the curb. In Boston only 7.7 per cent of 35,169 customers of retail stores who were interviewed used curb parking space. Interviews with 56,000 customers of large and representative stores in central Philadelphia showed 9.02 per cent to have used curb parking space. The parked vehicles in the central business district carried 2 per cent of the people entering that district."

The importance of parking therefore as an adjunct to the shopping business may be exaggerated. If parking were abolished absolutely not more than three Pittsburgh or eight Philadelphia shoppers in a hundred would be affected in being obliged to reach the stores in one of the other fashions of the remaining ninety-seven or ninety-two shoppers. For these communities, therefore, parking need not be protected in order to sustain the mercantile business of the central business district.

The elimination of parking accomplishes the following valuable results:

1. It enables vehicles at frequent space intervals to pass street cars, even on narrow streets, on the right; this being the correct side upon which to pass street cars, for safety reasons.
2. On three or four-lane streets, it permits horse-drawn vehicles or other slow-moving vehicles to pull to the right to permit street cars or other faster vehicles to pass.
3. It largely eliminates delays and decreases the hazard to moving traffic lanes, often including street cars, caused by vehicles "working their way" in or out of short parking spaces at the curb.
4. It is of great value to merchandise loading and unloading *where this must be done on the street* (it should be done off-street or in alleys as far as possible.)
5. It largely eliminates double-line parking, or loading and unloading from the second lane from the curb.
6. "No Parking" enables passenger vehicles, motor coaches, and taxicabs to discharge passengers at the curb, thereby not holding up lanes which should be exclusively for moving traffic. It thereby eliminates for passengers the hazard and discomfort of alighting in the middle of the street.
7. Because of the above-mentioned factors, it reduces congestion and enables a speeding-up of traffic movement, especially on

the more narrow streets. Increase in speed of mass transportation units means improved service to the great majority of the public.

8. Because of the speeding-up of traffic and the easy accessibility of loading entrances, trucking and deliveries are considerably reduced in cost.
9. The elimination of parking is of tremendous value to the Fire Department, both in moving along the street and in fighting a fire at such a location.
10. It reduces the hazard created by pedestrians walking out from between stopped vehicles.
11. It facilitates street cleaning considerably, and snow removal tremendously.
12. It greatly improves the effectiveness of display windows to people riding past in street cars, busses, or as passengers in automobiles.
13. Parking elimination makes congested business districts more accessible to the considerable majority of people who use public vehicles (including street cars, busses, and taxicabs) as compared to private automobiles. Business in general is benefited by the increased number of patrons who may conveniently enter and leave the district.
14. In important business districts, it encourages the development of off-street parking facilities so that even customers coming in private automobiles will find shopping more convenient and less time-consuming than with street parking permitted. The elimination of "hurried shopping" by customers fearing the end of short-time parking limits, is one of the benefits to merchants.

    These off-street parking facilities also make unnecessary the troublesome and time-consuming search for parking space by persons engaged in certain types of business which demand the use of automobiles intermittently during the business day.

    Off-street parking facilities also provide all-day, protected storage for those who still desire to use a private automobile in coming to the district.
15. "No Parking" reduces the number of stolen cars, a factor no longer unimportant. Even though in large proportion, stolen cars are recovered, they have usually suffered damage.

If it is desired to secure free and continuous traffic movement on the curb lane of any street where loading and unloading of pedestrians or merchandise must be expected, the solution is a NO STOPPING rule. This is a very drastic measure, which should generally be used only during peak hours, and then only where absolutely essential.

A somewhat less drastic rule, which is a partial further step toward continuous movement on the curb lane, is a regulation prohibiting parking and the loading and unloading of merchandise, but permitting loading and unloading of passengers. Such a rule of "NO STOPPING EXCEPT FOR PASSENGER LOADING" will not permit free and continuous movement on the curb lane. It will result in only short-time interruptions, however, and under certain conditions may prove desirable.

The committee believes that parking spaces, taxicab stands, and bus terminals should be considered as functions of business rather than a public responsibility, to be furnished off the public highway by those benefited.

SIGNALIZING OF MAIN ARTERIES

On heavy traffic urban streets where the traffic volume on most side streets is light, it is usually unnecessary to install traffic control signal at every intersection. A generally satisfactory method of control is to make the main street a Thru Traffic Street with traffic control signals at the more important intersections, and with effectively sign-posted compulsory STOP regulations for all other cross streets. The traffic control signals should be arranged in a flexible progressive signal system. As the distance between traffic control signals exceeds about 1200 to 1500 feet, satisfactory coordination is less and less to be expected. Therefore, some such distance probably represents the reasonable limit for expensive inter-connection. For greater distances coordination may be justified with synchronous motors which by design must operate at exactly the same speed if the same source of energy is used.

The above-described method of signalizing and stop-sign-protecting main arteries has, in general, the following advantages:

1. Through traffic will be attracted to these arteries because of protection against entering vehicles at all intersections, and because of the opportunity of progressive movement.
2. Pedestrains and vehicles in a given district are offered locations not too widely spaced apart where they may cross the major streets under protection of traffic control signals.

3. Even at non-signalized intersections, side street vehicles and pedestrains desiring to cross the major street, gain some advantage because of the general group movement on the major street.

It must be realized that the above statements are generalizations and that each case should be thoroughly analyzed to see if conditions exist which should alter the interpretation of these statements. Some such factors are: considerable proportion of slow-moving vehicles, wide roadways, bad paving conditions.

The committee strongly opposes the use of the traffic control signals spaced several intersections apart and intended to control traffic at intervening cross streets. This is a highly hazardous procedure; it also brings about the blocking of crosswalks by vehicles whose drivers must wait near the curb line in order to see the signal several intersections away.

INSTALLATION AND OPERATION OF TRAFFIC CONTROL SIGNALS

An attached chart (Table I) gives three methods of determination of minimum requirements for the installation and operation of a traffic control signal. These are under consideration of the Committee on Signs, Signals and Markings at Railroad Crossings and Major Highway Intersections of the National Conference on Street and Highway Safety.

Proposal No. 1 is stated to have been drafted by one of the larger western cities where extremely heavy traffic loads are encountered.

Proposal No. 2 is of Massachusetts origin.

Proposal No. 3 is a tentative standard and approaches the opinion expressed by the members of this committee.

The present Pittsburgh minimum standard which is believed by those in charge to be too liberal, provides for consideration of the installation of a traffic control signal when traffic volume is 600 vehicles per hour in the peak hour with at least 25 per cent on the lighter traffic street; also at least 250 pedestrians for the hour including the closing of school. If an intersection meets these minimum requirements such additional factors as the following are considered: need of speed control; school child protection; possibility and effect of inclusion in a progressive signal system; effect upon traffic routing, especially as to channelization of flow on main arteries and Thru Streets; intersection visibility; complications of grades, intersection layout, and traffic movements; traffic accidents; effect upon street car operation.

TABLE I

Standards for Rush Hour Operation

Proposal (No. 1)		Massachusetts (No. 2)	Tentative Proposal (No. 3)
A. Two roadways with two free lanes for movement in each direction.	1200 vehicles passing through it in maximum hour with 25% of this on the lighter travelled street.	A & B. (1) Vehicular Volume: Regardless of streets or lanes any signal may be operated during any hour when the total traffic is 500 vehicles per hour and the cross traffic is 125 vehicles per hour. (2) Pedestrian Volume: 500 vehicles. 250 pedestrians. (3) Speed Control: Special cases where excessive speed under uncontrolled conditions. (4) Coordination: Where majority of installations of system fulfill vehicular volume requirements and where it adds to the efficiency of the system. (5) Traffic Actuated: Anywhere, at any time, when properly timed.	(1) Vehicular Volume: Regardless of streets or lanes any signal may be operated during any hour when the total traffic is 750 vehicles per hour and the cross traffic is 200 vehicles per hour. (2) Pedestrian Volume. (a) 750 vehicles per hour. (b) 300 pedestrians per hour. (3) Speed Control: (a) 750 vehicles per hour, total flow. (b) Excessive speed under uncontrolled conditions. (4) Coordination: (a) Where majority of installations of system fulfill vehicular volume requirements; (b) Installation must add to the efficiency of the system. (5) Traffic Actuated: Anywhere, at any time, when properly timed.
B. Two roadways with three free lanes for movement in each direction.	25% more than the above set for two free lanes.		
Standards for All Day Operation			
C. Two roadways with two free lanes for movement in each direction.	16,000 vehicles pass in a 16-hour period and in addition has the rush hour standard.	Practically the same as above. If reduced to a total at 500 per hour, would be 6000 or ½ of the 16,000 required by the National Code Requirements.	Same as above.
D. Two roadways with three free lanes for movement in each direction.	25% more than above set for two free lanes.		
Standards for Non-Operation			
If traffic falls below 800 vehicles per hour, signals cannot be operated during such hours and an illuminated "Stop" or "Caution" sign should be displayed.		If traffic falls below 500 vehicles per hour. signals must be turned off from control operation and turned onto flashing yellow operation as a warning indication. If a through way, then red toward side street,	Same, except 750 vehicles.

VEHICLE ACTUATED TRAFFIC CONTROL SIGNALS

Until quite recently automatic traffic control mechanisms have been designed to provide continuously a pre-arranged, regular program or cycle. Some variations have been developed, but in general, the controllers give fixed percentages of the cycle to the different streets, these percentages being based on the average, or in some cases upon the peak hour, time requirements of traffic on the streets at the intersection.

Obviously, such a fixed program will not always prove efficient in time utilization. Sometimes traffic on one street will be held when there is no traffic using the " Go " period on another street.

Theoretically at least, it is evident that it is desirable for a signal to change its proportioning in accordance with changing traffic time demands. Where signals are not operating in a system, it is also desirable, within certain limits, that the cycle length change with changing time demands.

Such reasoning has brought about the development of several types of signal mechanisms actuated by vehicles approaching or waiting at the intersection. Although most, if not all, of these types of mechanisms are still in the development stage, the principle must be recognized as sound, although the field of practical application is, in general, considered to be limited. In general, these vehicle-actuated mechanisms include pedestrian push-buttons so that persons will not be required to wait for unreasonable periods before receiving a signal to cross. At certain intersections made up of more than two entering streets, properly designed full vehicle-actuated control covering all entering streets may have especial merit.

Some of the reasons why the field of application of most so-called vehicle-actuated signals is considered limited are:

1. In general, a traffic control signal is only considered warranted where and when traffic is quite heavy, and consequently the number of times when the regular program is inefficient is not unduly great. At least the inefficiency is usually not great enough to warrant the generally much greater cost of vehicle-actuated control.
2. Where flexible progressive traffic control signal systems are in use, all signals must operate on the same cycle length and a definite relationship must be retained between the starts of the " Go " periods at succeeding intersections. It is conceivable that some-

thing like a progressive system might be developed by interconnection of vehicle-actuated signals, and indeed some development has been made along this line. However, any such system would be complicated and costly.

3. The anticipation of rapid growth and stabilization of traffic at an intersection, or plans shortly to include the intersection in a flexible progressive signal system, will warrant careful consideration as to whether traffic actuated control should be installed.

TRAIN ACTUATED SIGNALS AT RAILROAD GRADE CROSSINGS

The necessity for uniform signs and signals for railroad grade crossings has been the subject of recommendations by this committee for several years. Since separation of railroad grade crossings can be accomplished only gradually, the committee believes that in order to facilitate and safeguard traffic movement, train signals actuated by approaching trains shall present to the highway traffic the appearance of the horizontally moving red light, the direction of which shall reverse alternately; the signals to be erected and function in accordance with the recommendations of the Signal Division of the American Railway Association, approved July, 1925.

Where a full view of these signals is less than 500 feet on account of curvature or grade in the highway, an additional signal consisting of a continuously flashing yellow light, embodied in the standard disc railroad approach sign (and so designed that the letters "RR" will be illuminated), shall be installed approaching the curve or grade so as to give advance warning of the curve and the railroad.

This committee further recommends that no road authority shall permit the erection of any horizontal alternately flashing red lights on the highways under their jurisdiction, except for the purpose of indicating the approach of trains (or interurban street cars) at grade crossings. The committee also recommends that the use of the standard disc approach sign shall be continued and additional ones erected as necessary, these approach signs to be used on unimportant roads where railroad and highway traffic is very light. The committee favors illumination of these wherever practicable. In cases where illumination is not practicable, the committee favors use of signs in which at least the letters "RR" are rendered luminous by automobile headlights.

LEFT TURNS

The committee believes that, since the capacities of two intersecting streets are limited by the capacity of the area within the intersection, the best turning method is the one that will permit the greatest movement within the intersection with the least interference with other movement.

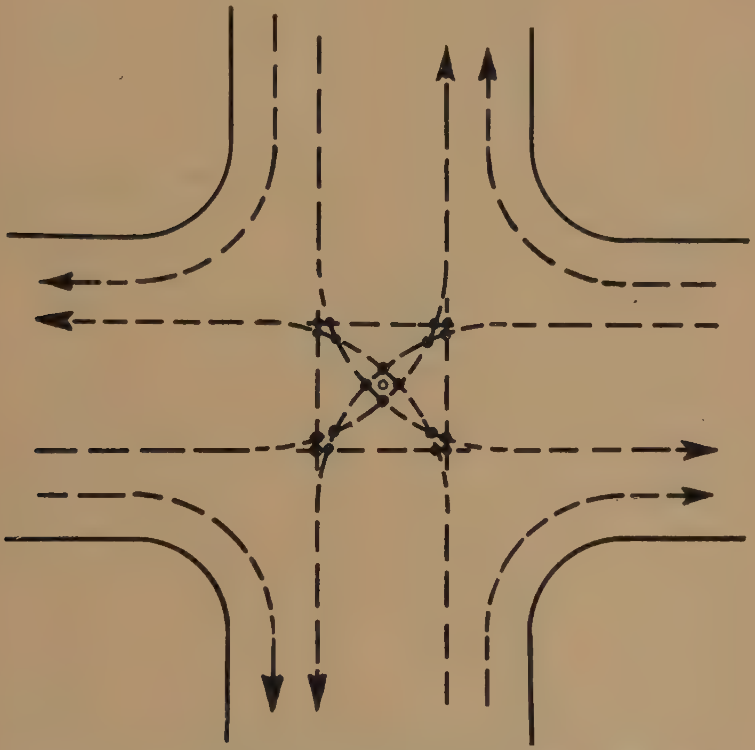


Figure 1. Uncontrolled Intersection—Left Turns Around Corner—12 Movements—20 Crossings

Vehicles approaching a normal intersection uncontrolled by lights or officers, from each of the four directions have the choice of three possible movements within the intersection; straight through, turn right or turn left. A total of twelve movements is possible.

The right turn away from the intersection causes little trouble or interference with other vehicles.

Through movement may be interfered with by cross traffic and left turns.

Left turns may intersect the paths of other turning vehicles as well as the through vehicles, and they therefore cause the most concern and greatest delay.

It is possible to make left turns in a variety of ways; a few of the most common are illustrated in Figures 1 to 8.

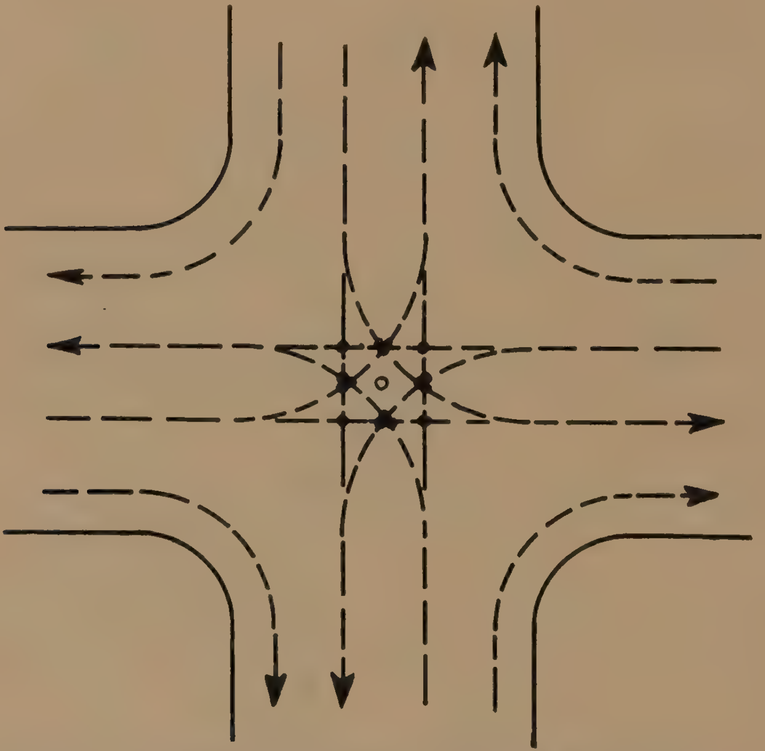


Figure 2. Uncontrolled Intersection—Left Turns Short of Center—12 Movements—16 Crossings

Figure 1 illustrates the movement in an uncontrolled intersection, vehicles turning left around the intersection of the center lines of the two streets. There are twelve movements with twenty points at which the paths cross.

Figure 2 illustrates an uncontrolled intersection, vehicles turning left short of, or " inside " of the center of the intersection. There are twelve movements with sixteen points at which the paths cross.

Figures 3 and 4 illustrate the movement in an intersection controlled by lights, giving a separate period for all left turns, usually the amber light, as well as the "stop" and "go" periods. In these cases the reservoir or storage space for left turning vehicles is at the right curb, occupying and obstructing the paths of through vehicles and those turning right. The throat of the intersection is obstructed and part of the street area lost. The light cycle is split into three parts and the left turns, when made, permit only four movements within the intersection. The paths cross at eight points when the turns are made around the center, and at four points when the turns are made short of the center, as in Figure 4.

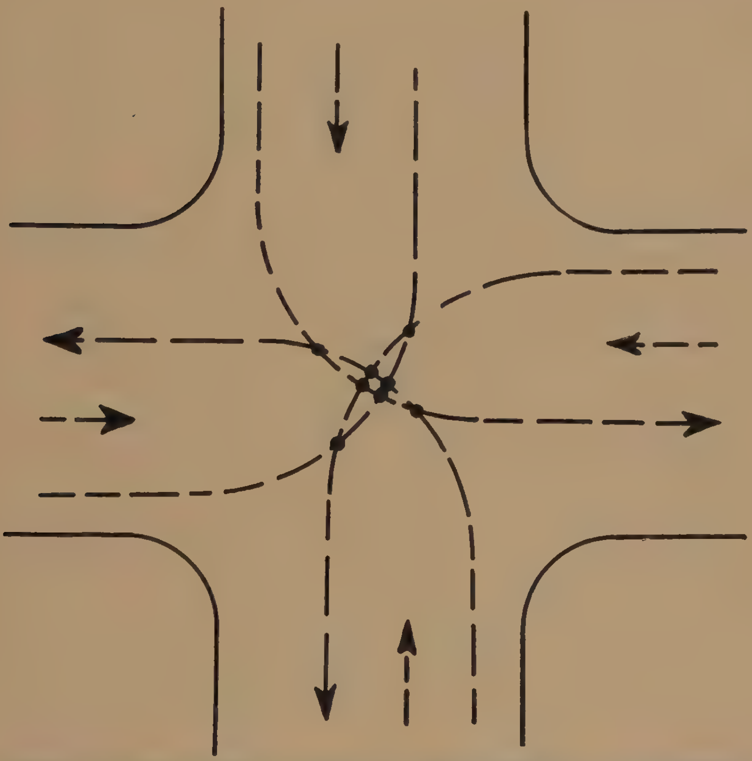


Figure 3. Left on Special Light—Around Center—4 Movements—8 Crossings

The facts remain the same when the turns are made from the lane of traffic to the right of and nearest the center lines of the streets instead of from the position at the curb. In both cases vehicles with long wheel bases will be unable to execute the movement if forced to pass around the center of the average intersection instead of being permitted to turn inside such center.

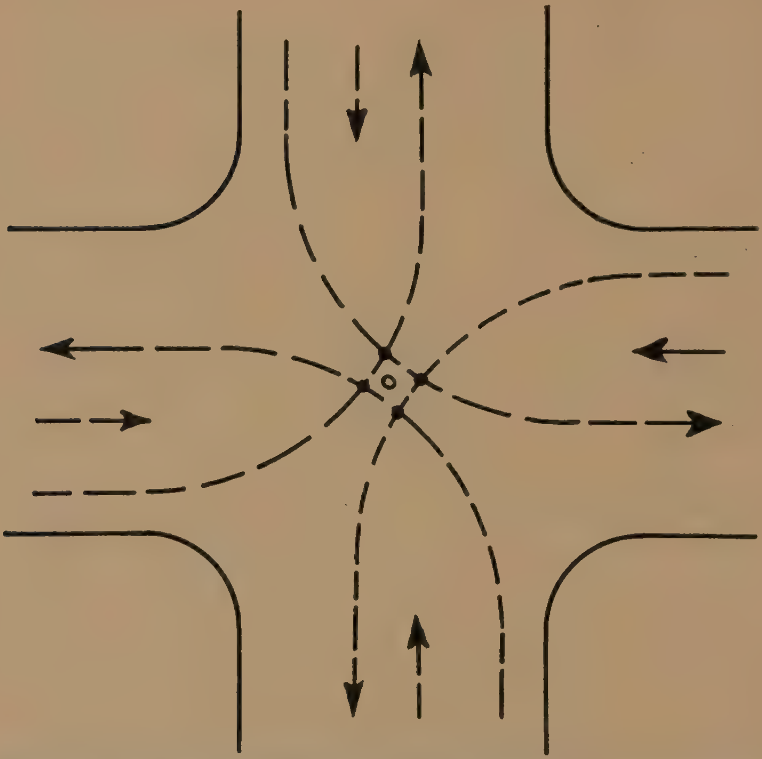


Figure 4. Left on Special Light—Short of Center—4 Movements—4 Crossings

The most simple method of controlling traffic at an intersection is undoubtedly the best, provided that the maximums of movement and safety are assured. Certainly nothing is more simple than to permit traffic to proceed on one street for a time while the other is stopped and then reverse the procedure.

Figure 5 illustrates the free movement on one street, including right and left turns, while all movement on the intersecting street is stopped. In this case, left turns are made from the lane of traffic next to and to the right of the center line of the street, and pass around the center of the intersection. There are six movements taking place within the intersection, and the paths cross at four points.

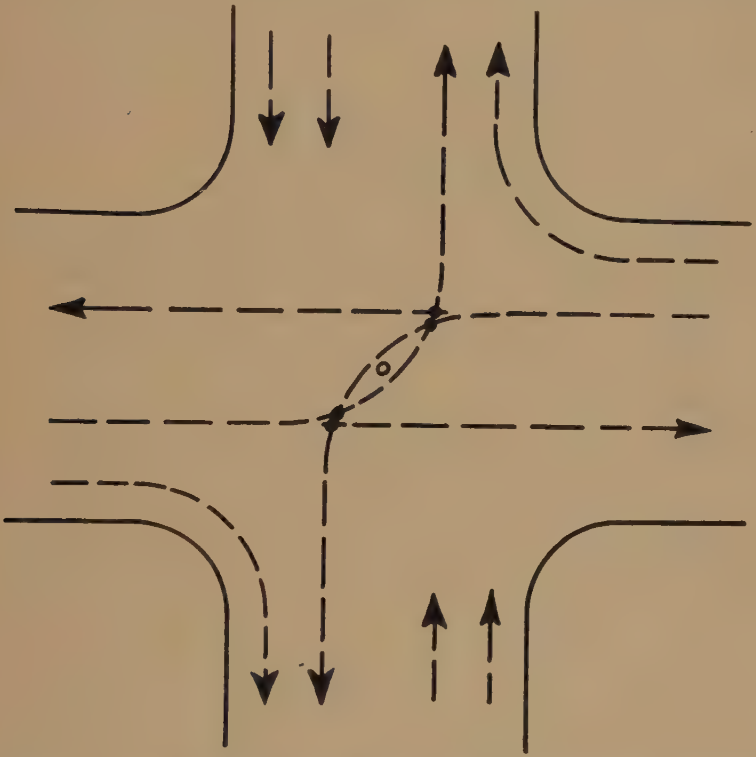


Figure 5. All Turns on Green Light—Left Turns Around Center—6 Movements—4 Crossings

Figure 6 illustrates the same features except that left turns are made inside of the center of the intersection. There are eight movements taking place, the paths crossing at four points. With six movements the paths would cross at only two points.

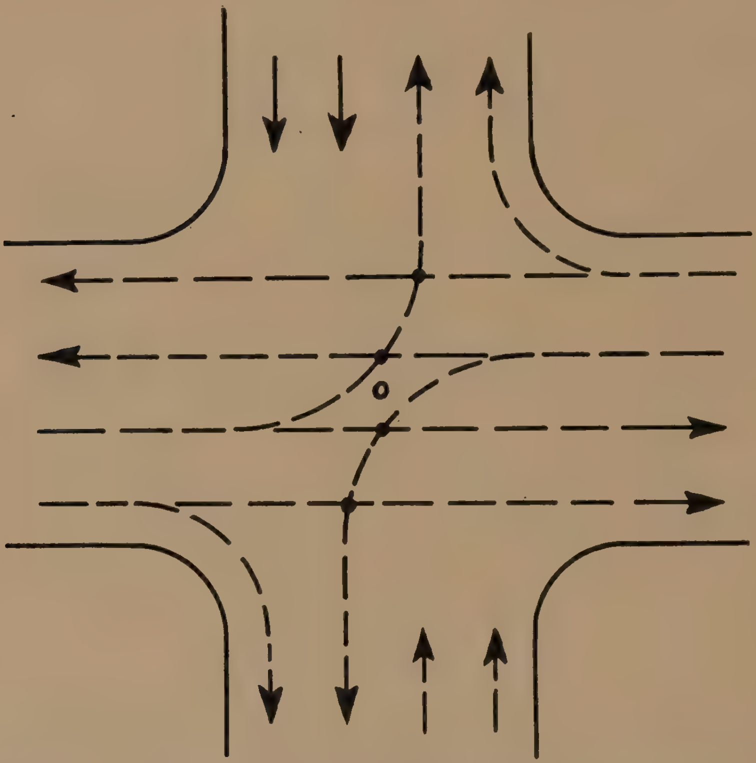


Figure 6. All Turns on Green Light—Left Turns Short of Center—8 Movements—4 Crossings

Figure 7 illustrates what is popularly known as the "left turn on the red," recommended by the committee as undesirable and counter to general practice. In this case the same narrowing of street width is encountered due to vehicles stopped before the green light awaiting the red light to make the turn as are to be found in illustrations three and four, where the special turning period is inserted in the cycle. It is possible for eight movements to take place within the intersection with six points where the paths cross.

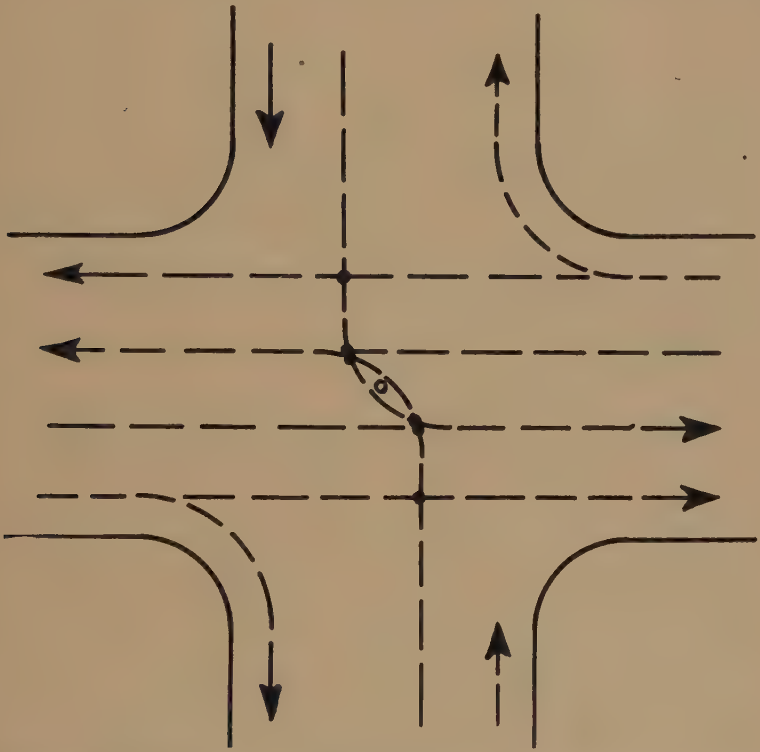


Figure 7. Left Turns on Red Light—Around Center—8 Movements—6 Crossings

At intersections of streets wide enough for only one lane of traffic in each direction, no space is available for the storage of vehicles awaiting a special turning period. All turns must be made on the green or "go" signal.

Figure 8 illustrates what is known as the "rotary left turn." It is adapted only to unusually wide streets and under certain other favorable conditions. It has the apparent advantage that free movement through the intersection is possible without any interference from left turns. There are, however, several important objections to the method:

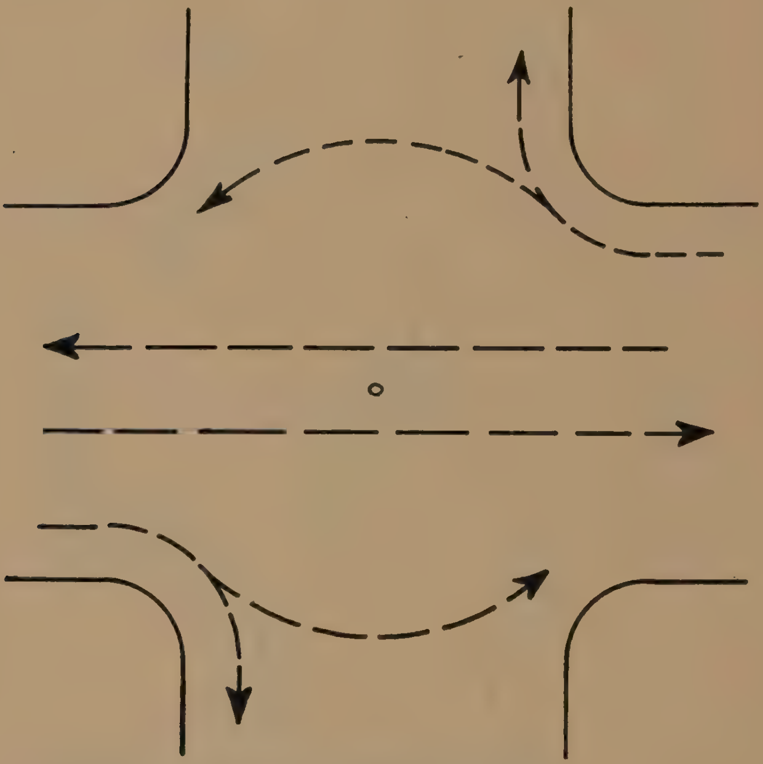


Figure 8. Rotary Left Turns

1. It is unusual, not in accordance with general practice, and therefore not easily understood by strangers; hence it causes confusion and delay within the intersection.
2. Vehicles waiting to turn left may block the crosswalk. It also leaves the pedestrian in uncertainty. He does not know whether to jump forward to avoid a right turn or backward to avoid the rotary turn.
3. The width of the intersecting street may prove insufficient to provide storage space for all left turning vehicles, resulting in delay.

4. If the number of left-turning vehicles is large, the vehicles to turn left block either or both the right turning vehicles and the straight moving vehicles.
5. On streets with car tracks, a lag in starting the rotary movement when the signal changes will result in a street car moving across to be blocked by the rotary movement; the result being that vehicular traffic is stalled in both directions and the intersection congested for a time.

The method illustrated in Figure 6 is clearly demonstrated to most completely meet the requirements, *i. e.,* maximum freedom of movement with least interference and possibility of universal adoption. This method is also in accord with the normal turning procedure at an uncontrolled intersection.

Therefore, the committee recommends that left turns be made on the green or " go " signal, from the lane of traffic immediately to the right of and nearest to the center of the street; the turning vehicle passing inside or to the left of the center of the intersection and as closely as possible thereto.

THEORETICAL LIMITS TO TRAFFIC CAPACITY

The theoretical capacity of a highway may be determined for an assumed uniform condition of traffic flow. To get the maximum flow of vehicles uniform speed and a constant distance apart must be assumed. Observations on the highway indicate that drivers space themselves in proportion to the square of the speed of travel. This is as it should be because the distance necessary to stop a moving vehicle is proportional to the square of the speed of travel. Assuming a distance from the front of one vehicle to the front of the next vehicle when standing still as 15 feet, assuming four wheel brakes with a coefficient of fraction of 0.6 and assuming one-half second reaction time, we have the following equation for the theoretical capacity of one lane:

$$N = \frac{5280Y}{0.73Y + 0.056\ Y^2 + 15},$$

where N is the number of vehicles per hour and Y is speed in miles per hour. It will be noted that the capacity varies with the speed—the maximum capacity coming with a speed of 16.5 miles per hour.

When the travel is heaviest the speed drops down to that of the slowly moving vehicle. If there are no horse drawn vehicles or very heavy trucks on the road, the capacity of one lane will actually approximate the theoretical capacity. The speed of movement for the faster vehicles will be decreased but the actual discharge will be more than can be expected if the average speed is 30 miles per hour.

When vehicles are moving at different speeds making it necessary for some of them to pass others going in the same direction, the problem becomes more complicated. If the highway has two lanes, the opposing lane must be free at the time of passage. This cuts the capacity of the oncoming lane and if the number of slower vehicles is large it may reduce the carrying capacity of the second lane to a minimum.

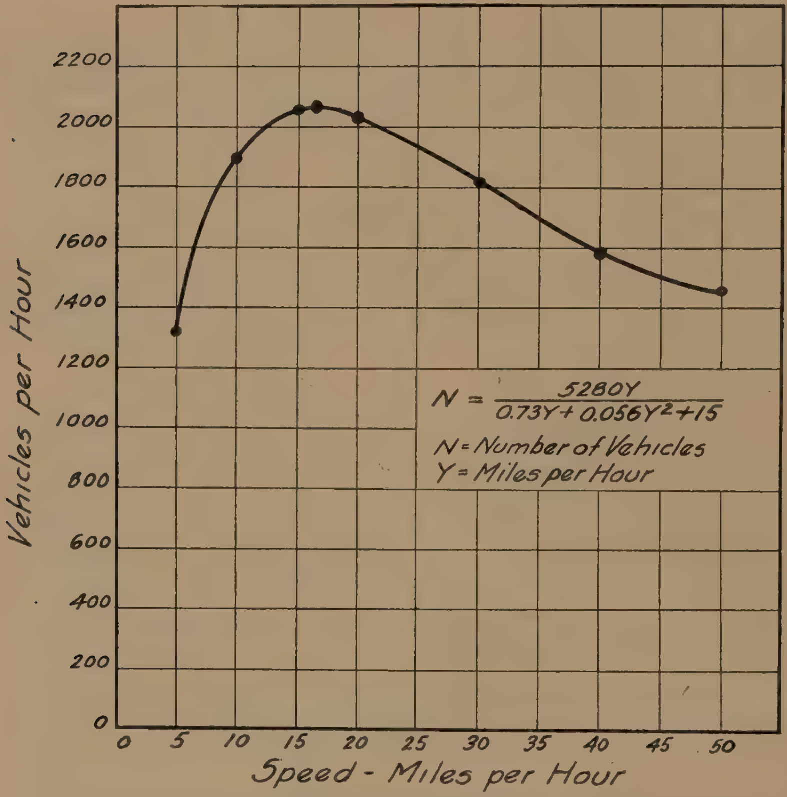


Figure 9. Capacity of One Lane at Different Speeds, Assuming Four-Wheel Brakes

A third, or passing lane, may be used to advantage if the mixed travel is predominantly in one direction during the peak flow. Here is introduced a danger element which makes the three lane road questionable for a balanced flow of mixed travel. If the balanced flow is more than the capacity of two lanes, four lanes are needed.

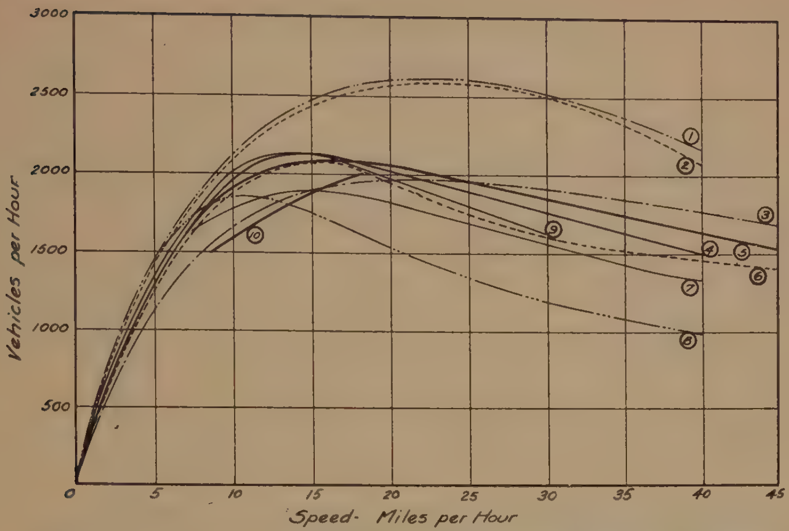


Figure 10. Comparison of Curves Showing Maximum Hourly Capacity of a Single Traffic Lane at Different Speeds. Data from a Number of Sources

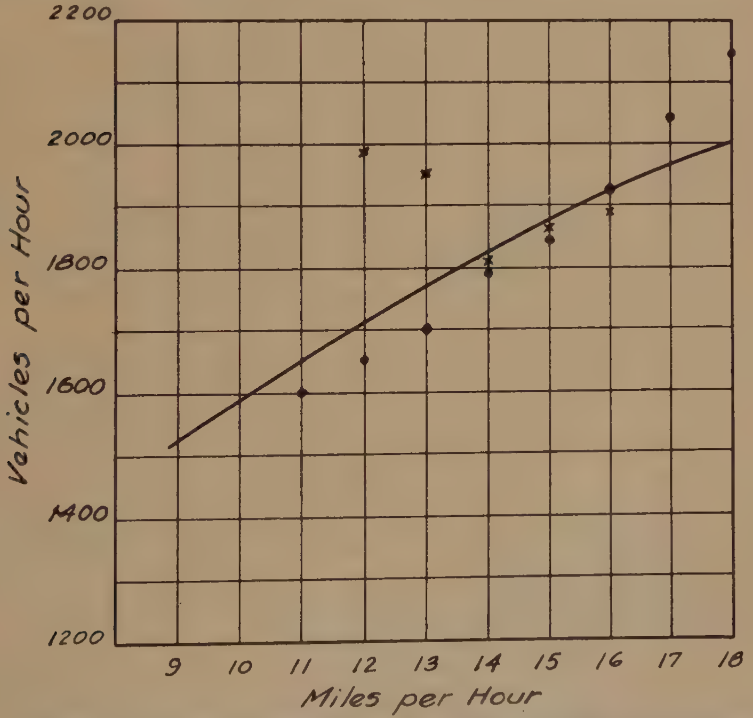


Figure 11. Hourly Capacity, One Free Wheel Traffic Lane. Bureau of Traffic Planning, Pittsburgh, Pa. Based upon Studies in Downtown Pittsburgh. Figures Given Are Theoretical, Representing the Number of Vehicles There Would Be in a Full Hour's Flow of Traffic Moving in a Flexible Progressive Signal System. For Actual Capacity Lane Flow at an Intersection, the Percentage of "GO" Time on the Street in Question Must Be Applied to the Graph Figure at the Proper Speed. Example; 45 Per Cent "GO" Time on "A" Street, Average Speed of Traffic Through Intersection 16 Miles Per Hour, Actual Lane Capacity Through Intersection Equals 0.45 × 1925 Equals 866 Vehicles

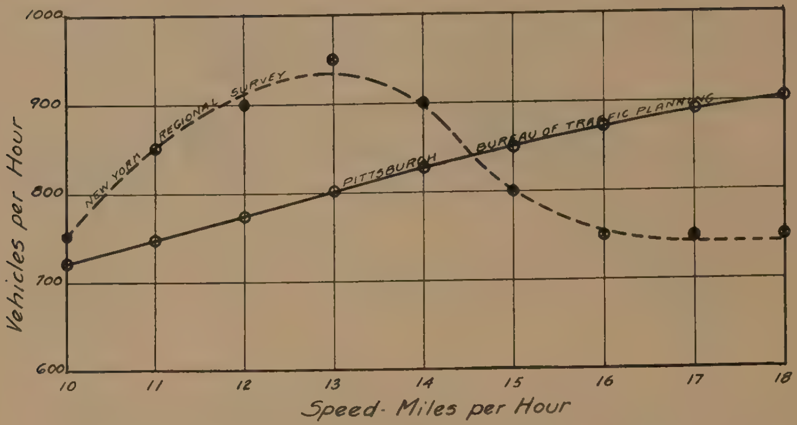


Figure 12. Comparison of Lane Capacity Data, Showing Curves for New York Regional Survey and Bureau of Traffic Planning, Pittsburgh, Pa. Both Curves Are on the Basis of a Cycle of 53 Seconds "GO" Time of 45.3 Per Cent of the Cycle or 24 Seconds

MEASURE OF TRAFFIC CONGESTION

Congestion—Hindrance to Movement of Vehicles over Highway or Street due to other Vehicles

1. Congestion is a relative term depending upon the kind of road and the area which it serves. A vehicle is not expected to move at the same speed in the "Loop District" of Chicago as on the "Speedway" at Indianapolis.

The committee believes that vehicular congestion exists.

(a) On rural highways: Where, because of other vehicles, speed is frequently less than 30 miles per hour.

(b) On urban highways: In residential areas where, because of other vehicles, speed is frequently less than 20 miles per hour.

(c) In business districts where speed, because of other vehicles, is frequently less than 12 miles per hour.

2. Methods of studying traffic movement. Forms of equipment will vary with the information sought. Most elementary studies may be made using a stop watch and ordinary speedometer. More careful studies may be made by using self-recording time distance instruments.

TRAFFIC LANE MARKINGS

(a) *Urban.* Any street having more than two lanes in one direction will have its effective capacity reduced by irregular lateral spacing of vehicles. The effective designation of lanes aids drivers materially in keeping in place. It also aids in reducing accidents.

Lane lines may be constructed with the pavement by the use of longitudinal construction joints or by the insertion of markers. Where lane lines are not constructed with the pavement, the most common method of marking is by use of paint. In order to reduce cost and to differentiate them from continuous center lines, lane lines are generally broken or dashed lines with considerable spaces between the dashes.

Although general lane marking is desirable, the expense of marking can be reduced if the lines are carried only adjacent to intersections, opposite safety zones, on curves, and at other points where it is especially important for the expedition and safety of traffic that vehicles keep within their own lanes. Special directional arrows can sometimes be used effectively to supplant lane lines, as for left turns inside of the center of the intersection or for other routings not easily indicated otherwise. Where continuous center lines are used, some effectively different marking should be used at locations where traffic is not to cross the center line, as on sharp horizontal and vertical curves and at railroad grade crossings.

Where paint is used, extensive tests by the Pennsylvania and Connecticut Highway Departments indicate that white is the most effective color. A four-inch width is being successfully used and is believed to be practically as effective as the six-inch line formerly used.

In order to eliminate the considerable cost of repainting, much experimentation has been carried on to find satisfactory "permanent" insert markers. Numerous obstacles have been encountered. Where snow plows are used, all markers must be practically flush with the pavement surface, for the impact of the blade against any rigid obstruction will nick or shatter the blade and interfere with the snow removal operation. In any case, markers should not be raised above the surface more than ½ inch and they should have rounded edges, so as to present a better surface to hard tires. Markers must also be firmly imbedded in the pavement, or they will be worked loose and finally removed by vehicles. There is a rather difficult problem of securing effective night visibility. This problem is reduced where there is overhead illumination or practically continuous traffic in both directions. The marker must always present an easily visible contrast to the pavement surface—they must not become dull in service. For any extensive use, the markers must be reasonable both in purchase price and installation cost. Several types of markers have been developed which are reasonably effective for at least certain pavement surfaces.

(b) *Rural.* On rural roads it is probable that lane marking will be valuable at intersections or in semi-congested conditions. In general, however, the committee believes it is unnecessary to lane mark four lane highways, and that a slight amount of traffic control and supervision will keep drivers in their proper positions.

Most of the discussion for urban streets applies for rural highways. The statement about snow plows and the problems of night visibility are especially applicable. A flat paint has been found to reflect more light from headlamps than a glossy finish.

RESPONSIBILITY FOR PROVIDING FOR PEDESTRIAN TRAFFIC

The existing condition is that on some sections of rural road, especially where the roadside development is intermediate between rural and urban, at the outskirts of large communities, for instance, or running through a small village, there is an amount of pedestrian traffic which, with high-speed motor vehicle traffic, introduces a large element of hazard, especially at night.

The difficulty is that in such cases the local community does not have adequate means for providing sidewalk facilities. The historical phase is that the road was utilized with comparative safety for pedestrian traffic some years ago, but that the road surface has been improved and traffic has increased and has been speeded up, so that now the road is, with consideration to safety, impracticable for pedestrian use.

The roads are being built and maintained with motor vehicle funds, license fees and proceeds of the gas tax, and the general impression is that the motorist should not be expected to provide facilities for the pedestrian, but justice would seem to indicate that the motorist having pre-empted facilities formerly enjoyed by the pedestrian, should provide substitute facilities. This is the logic of the principle by which a railroad company, laying track within an established highway right-of-way is required to provide at its own expense a road equal to the one that it has taken over. Similarly, State Highway Departments must soon recognize the responsibility for taking care of pedestrian traffic and provide adequate and safe footpaths at locations where such traffic is sufficient to warrant the expenditure.

VALUE OF ESTABLISHING INDEPENDENT HIGHWAY ROUTES FOR HIGH SPEED AND LOW SPEED TRAFFIC RESPECTIVELY

1. Congested areas.
2. Industrial areas.

This topic is of particular interest in city and zone planning but may also be of interest to some of the State Highway Departments in the near future.

The anticipated condition is that on certain roads, particularly in and in the vicinity of large cities, there may be such a volume of low speed truck traffic that the usefulness of the road to passenger motor vehicles will be seriously impaired.

Under this circumstance, the most apparent relief is the widening of the road surface to provide an outside lane in each direction of traffic for the low-speed trucks. However, in case the combined volume of passenger vehicles and trucks exceeds the practicable carrying capacity of the 4-lane road, local conditions may prevent the further widening of the road, right of way for instance in or entering a community where the properties are built up in such way that extra widening would require excessive expenditure. In such cases relief might be provided by construction of parallel road. If along considerable lengths of either the original or the parallel road, there are no sources or destinations of truck traffic, or if access to the other road may be provided to such truck traffic as may originate or be destined within such sections, the condition would appear to be favorable to segregation of the trucks on one of the roads, leaving the other for the sole use of passenger or high-speed motor vehicle traffic. There should be no objection to the use of the truck road by such passenger vehicles as might be willing to accept the unfavorable operating conditions of that road. Segregation of the low-speed traffic as well as facilitating the movement of traffic on the other road offers the possibility of reducing the maintenance costs and increasing the durability of the surface utilized by only light-weight vehicles.

Pennsylvania Department of Highways records, at the present time, do not show any road sections where truck traffic is of sufficient volume to warrant construction of separate roads.

FORMS FOR TRAFFIC INVESTIGATIONS

The forms submitted herewith are illustrative of current practice and are presented as basis for a uniform system.

It is the intention of the committee to conduct a further study of traffic forms during the coming year, and to include definite recommendations in the next report.

Form Number 1. This is the form used where it is unnecessary to list the traffic separately by the direction in which it is moving, or by right and left turns. The directions at the top of the columns indicate the directions of the road from the intersection. For instance, at the intersection of two roads (commonly known as a four-way intersection) there are roads to the north, south, east and west. The total passenger car traffic then on the north road would be the amount entering the intersection from the north road plus the amount leaving the intersection by the north road. Counting traffic on this basis involves counting the vehicle once on the road by which it approaches the intersection and checking it again on the road by which it leaves the intersection. It will be noted that at the top of the sheet there is a space for station number. In a large survey it is essential that for ease of reference and for general efficiency each station be given a number, the roads involved at this station being listed on each density sheet as well as on a control sheet held in the office.

Form Number 2. This form is suggested for use at intersections where the volume of traffic is small but where a record of the distribution of traffic by directions and turns is desired. The form provides space for 12 directions, the number of sheets required depending upon the period of the count and the time interval used. It is assumed that one man can record all traffic and that the hand tally method of counting is employed.

Form Number 3. This form is suggested for use at heavy-traffic intersections in cities. It provides space for four directions, the maximum number which one man can handle where the volume of traffic

(NAME OF ORGANIZATION)

TRAFFIC DENSITY

Station No.________ Day________ Date________ Hours Scheduled________

North Road________ East Road________

South Road________ West Road________

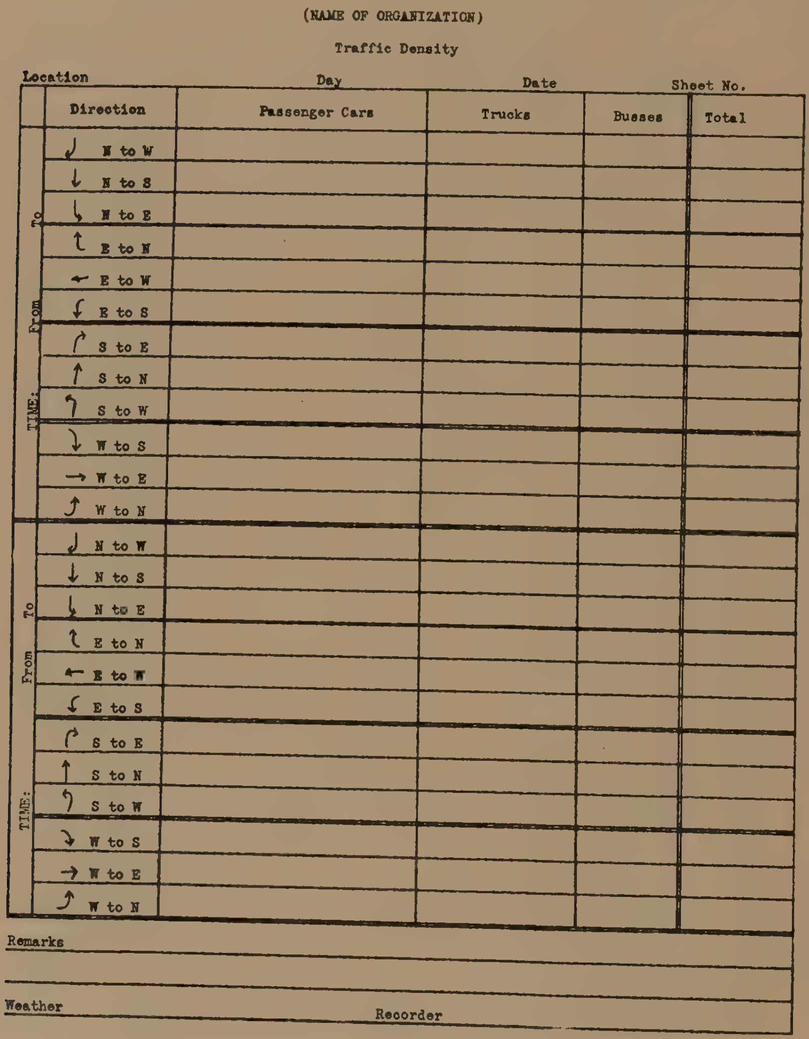
Hours	Passenger Cars				Trucks				Busses				Total			
	North	South	East	West	North	South	East	West	North	South	East	West	North	South	East	West
6- 7 A.M.																
7 -8																
8- 9																
9-10																
10-11																
11-12																
12- 1 P.M.																
1- 2																
2- 3																
3- 4																
4- 5																
5- 6																
6- 7																
7- 8																
8- 9																
9-10																
Total																
Foreign																

Remarks________

Weather________ Recorders________ Party No.________

Form Number 1

is great. In many cases, particularly at an important four-way intersection, only three directions of traffic can be counted by one man, in which case the last bloc on each sheet is not used.



(NAME OF ORGANIZATION)

Traffic Density

Location Day Date Sheet No.

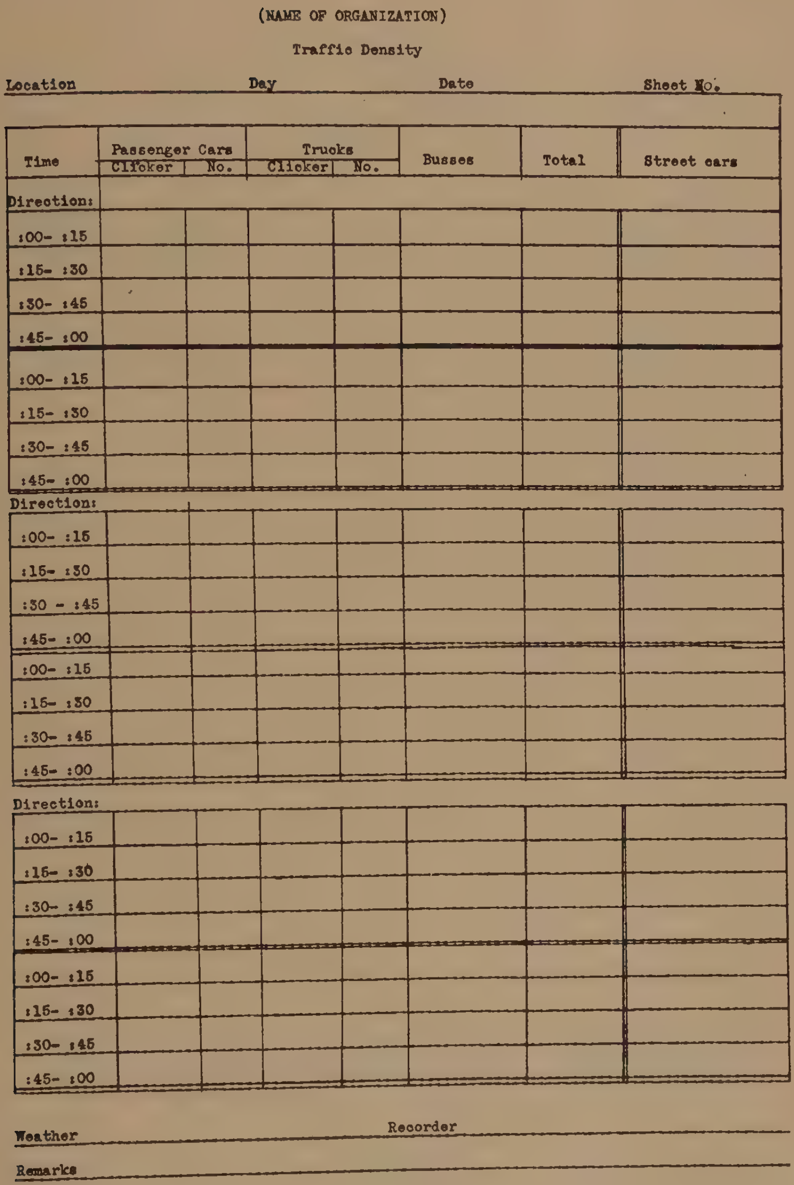
	Direction	Passenger Cars	Trucks	Busses	Total
TIME: From To	↲ N to W				
	↓ N to S				
	↳ N to E				
	↰ E to N				
	← E to W				
	↲ E to S				
	↱ S to E				
	↑ S to N				
	↰ S to W				
	↳ W to S				
	→ W to E				
	↱ W to N				
TIME: From To	↲ N to W				
	↓ N to S				
	↳ N to E				
	↰ E to N				
	← E to W				
	↲ E to S				
	↱ S to E				
	↑ S to N				
	↰ S to W				
	↳ W to S				
	→ W to E				
	↱ W to N				

Remarks

Weather Recorder

Form Number 2

As the form indicates, passenger cars and motor trucks are counted by means of a tally counter or clicker. The readings recorded from the clicker should be cumulative. The recorder, during the course of the count, should not set the clicker back to zero. Space is provided to permit the hand tallying of motor busses and street cars.



(NAME OF ORGANIZATION)

Traffic Density

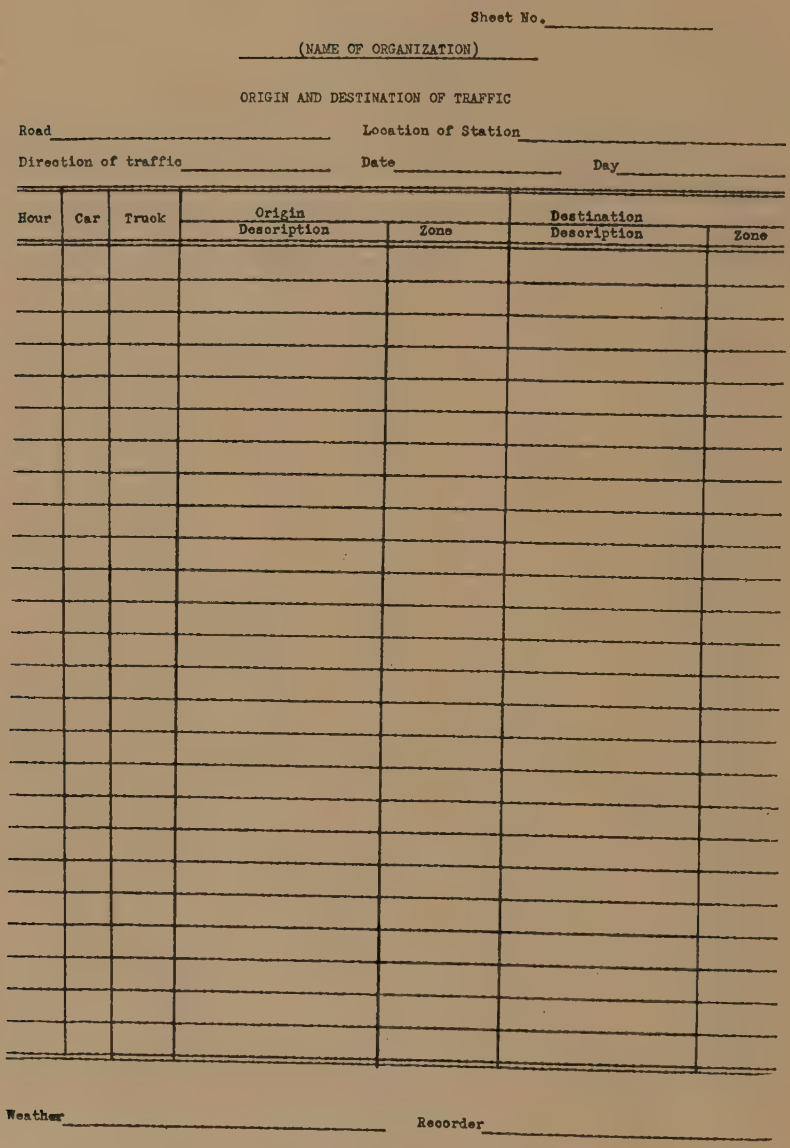
Location Day Date Sheet No.

Time	Passenger Cars		Trucks		Busses	Total	Street cars
	Clicker	No.	Clicker	No.			
Direction:							
:00- :15							
:15- :30							
:30- :45							
:45- :00							
:00- :15							
:15- :30							
:30- :45							
:45- :00							
Direction:							
:00- :15							
:15- :30							
:30 - :45							
:45- :00							
:00- :15							
:15- :30							
:30- :45							
:45- :00							
Direction:							
:00- :15							
:15- :30							
:30- :45							
:45- :00							
:00- :15							
:15- :30							
:30- :45							
:45- :00							

Weather Recorder

Remarks

Form Number 3



Sheet No.________________

(NAME OF ORGANIZATION)

ORIGIN AND DESTINATION OF TRAFFIC

Road____________________ Location of Station____________________

Direction of traffic____________ Date____________ Day____________

Hour	Car	Truck	Origin		Destination	
			Description	Zone	Description	Zone

Weather____________________ Recorder____________________

Form Number 4

If more than four directions can be handled by one man, a modification of Form No. 2 with a column for street cars and with or without "clicker-reading" columns can be used.

Form Number 4. This form is used in the origin-destination study, when vehicles are stopped and the points of origin and destination of the trip noted. Under the car column is marked the time of starting and the beginning of each hour thereafter. The hour should also be marked at the top of each sheet. If the vehicle stopped is a passenger car, a check mark is put in the car column, otherwise a check mark is put in the truck column. The description of the origin or destination is then marked down after the driver has been questioned. A definite list should be made up in advance showing the groups into which it is desired to have the origins and destinations recorded. For instance, if the origin or destination is outside of the state, it is unnecessary to obtain very minute description; probably the state or county might be sufficient. If the area to be included in the survey is a particular county, all points outside of that county might be listed, as in a particular county, with the exception of adjacent counties, which might be divided into two or more parts. In the county of the survey, however, it is necessary to obtain a more detailed description, such as the village or township. If the survey is in or adjacent to a large city, the city address or the nearest street intersection to this address should be taken and recorded in the description column. The column headed "zone" is provided for office work. The field records, after receipt in the office, are checked and the various descriptions given zone numbers which have been set up in advance. If a city is divided into a number of zones for the purpose of the origin-destination study, each zone is given a zone number which is used to classify the various descriptions listed by the field recorders.

SEASONAL DISTRIBUTION OF TRAFFIC

A. N. Johnson,

Dean of Engineering, University of Maryland

The traffic counts taken by the State Roads Commission of Maryland for the past twelve years at over two hundred points furnish the data for this discussion.

Since 1917, traffic data have been secured over the entire state highway system, which in that year was fairly complete, that is, all the main through roads had been built, and were in use.

The traffic counts were taken one day per month from 6 a. m. to 6 p. m.; thus, any analysis or conclusions are subject to the limitations this quantity of data imposes.

One of the studies made on these data was to determine the seasonal variation or distribution of traffic throughout the year. Three points: Baltimore, Frederick, and Salisbury were chosen for comparative study. These particular points were selected partly because of the completeness of the traffic records in the vicinity, but principally because of their geographic and economic location; Baltimore being a large commercial and industrial center; Frederick lying in the central part of the state in a fertile valley, and Salisbury being representative of the truck farm districts on the flat sandy Eastern Shore country.

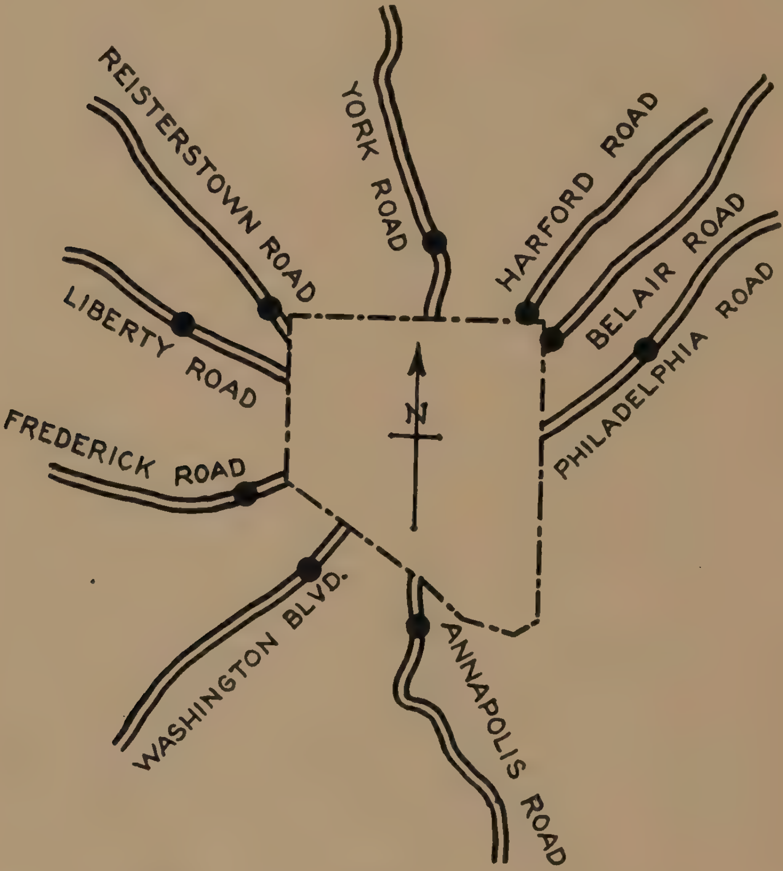

REISTERSTOWN ROAD
YORK ROAD
HARFORD ROAD
BELAIR ROAD
PHILADELPHIA ROAD
LIBERTY ROAD
FREDERICK ROAD
N
WASHINGTON BLVD.
ANNAPOLIS ROAD

Figure 1. Traffic Stations Vicinity of Baltimore

In the vicinity of each of these cities the traffic stations were located as shown on Figures 1, 2 and 3, and the average daily traffic at each of these stations for the years 1926 to 1928, inclusive, was obtained. The average daily traffic for all the stations around a given center for the past three years was taken at 100 per cent, then the average daily traffic for all the stations for each month for a three year period was ascertained, that is, all the Januarys for each of the years was averaged and called the January traffic. The percentage that this average traffic for each month was above or below the yearly average was computed and platted, as shown on Figure 4.

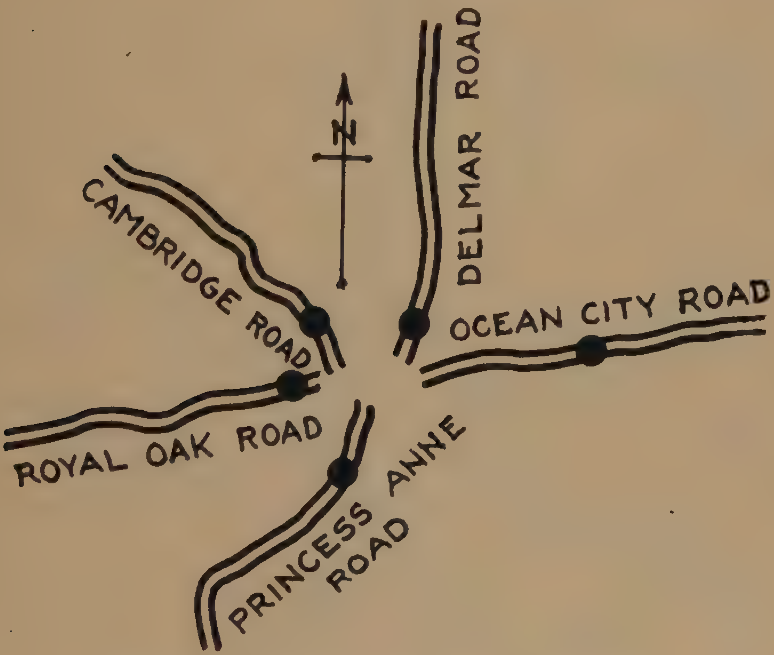


Figure 2. Traffic Stations Vicinity of Salisbury

It will be noted there are two curves on each diagram; one, for the years 1926 to 1928, inclusive, which was based upon the data as here described. The second or dotted curve is the average daily traffic for the years 1917 to 1920, inclusive, and was made in a similar manner, but was not based upon as complete data, in that the curves for 1917 to 1920 included but two stations, and are therefore subject to greater inconsistencies. But the general trends are concordant and we can accept the results as giving a very fair portrayal of the variation between the traffic ten to twelve years ago and that of today.

Thus, about Baltimore, in 1917 to 1920, the January traffic was about 50 per cent of the average yearly traffic, while in September it was 160 per cent of the average yearly traffic, a variation of 110 per cent.

About Salisbury, similar data show that January traffic was 58 per cent of the yearly average, while the August traffic was 160 per cent of the same, a variation of 102 per cent.

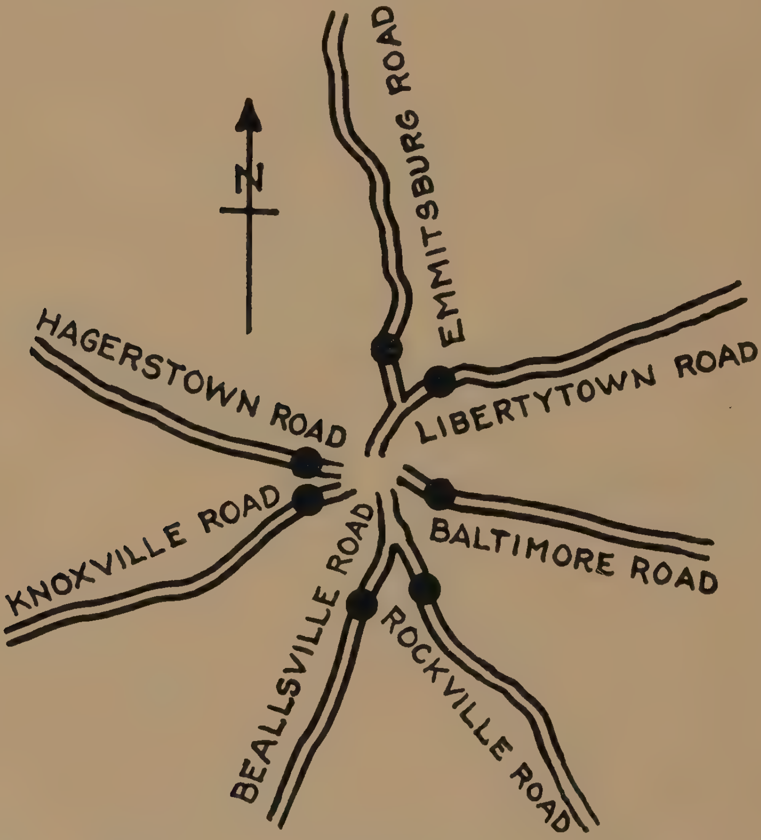

N
EMMITSBURG ROAD
HAGERSTOWN ROAD
LIBERTYTOWN ROAD
KNOXVILLE ROAD
BALTIMORE ROAD
BEALLSVILLE ROAD
ROCKVILLE ROAD

Figure 3. Traffic Stations Vicinity of Frederick

Again, around Frederick it is noted that the average traffic in January ten years ago was but 29 per cent of the yearly average, while the maximum in October was about 166 per cent of the yearly average, a range of 137 per cent.

These variations are in accord with the relative severity of the winters in these neighborhoods.

We now note the solid curves on the diagrams which show the seasonal variation during the past three years, 1926 to 1928, inclusive.

The minimum about Baltimore, in February, is seen to be 79 per cent of the yearly average, while the maximum month is 121 per cent, showing a difference of but 42 per cent, as compared with 110 per cent ten years ago.

About Salisbury, we notice a similar trend for the past three years, that is, the difference between the maximum and minimum traffic months is about 52 per cent, as compared with 102 per cent ten years ago.

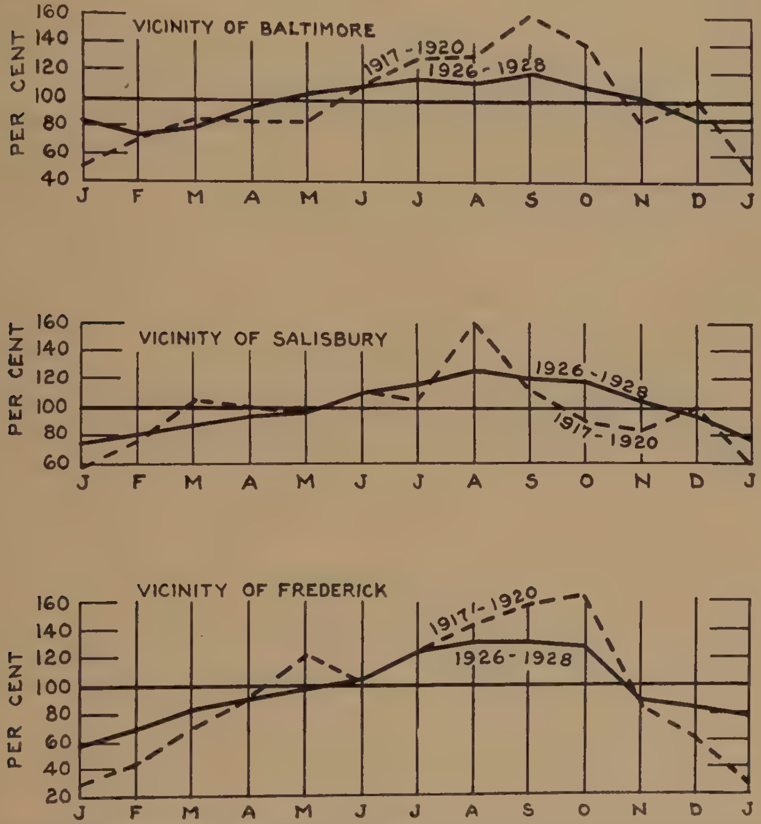


Figure 4. Monthly Variation of Traffic. Average Traffic Throughout Year Taken as 100 Per Cent

About Frederick, for the past three years the variation is 76 per cent as compared with 137 per cent ten years ago.

Not only has the amount of traffic during bad weather increased, but relatively this amount is a greater proportion of the traffic than was the case ten or twelve years ago. Then the roads, during the winter months, accommodated but a third of the average traffic throughout the year, whereas, today the traffic during the bad weather months is nearly three-fourths of the yearly average.

This change, without doubt, has been brought about mainly through the general adoption of closed cars, which make it possible to ride comfortably in stormy weather. No doubt, too, the better attention given to our roads, particularly the removal of snow, has helped to increase the use of road during bad weather.

Another set of data illustrating the relative use made of roads during fair and stormy days was collected at a given point on the Baltimore-Washington road near the University of Maryland. Traffic was counted on a stormy day for a period of one hour. Suppose this happened to be on a Wednesday. Then the next fair Wednesday, at the same hour, traffic would be counted again at the same point. Seven such comparisons were made on different days during March and April, 1929. The ratio of rainy weather to good weather traffic varied from a minimum of 67 per cent to a maximum of 111 per cent; in the latter case it happened that the hour counted for the rainy weather was in excess of that of a similar count of the fair weather traffic. The average, however, was 87 per cent.

With the continued improvement of vehicles and road surfaces, it may be expected that the difference between the use of our roads in good and in bad weather will continue to decrease, and that highway departments are amply justified in employing every practical means for the improvement of road surfaces during bad weather.

DISCUSSION

ON

THE REPORT OF THE COMMITTEE ON HIGHWAY TRAFFIC ANALYSIS

William H. Connell, *Executive Director, Regional Planning Federation of the Philadelphia Tri-State District.* Before entering into a discussion of highway traffic analysis, I want to compliment the committee on what I consider an exceptionally good and comprehensive report on a very important subject. Instead of discussing the recommendations in the report, and the subject matter in detail, I will confine my remarks to pertinent observations relative to the subject. I will also submit for your information a discussion of a very interesting Origin and Destination Traffic Survey made by the Regional Planning Federation of the Philadelphia Tri-State District in June, 1929.

There has been considerable discussion relative to the advisability of having traffic engineers in charge of traffic divisions in all communities and cities where the problem is of sufficient importance, and in all state highway departments. To my mind this suggestion does not go quite far enough. A traffic division is important but there should also be a planning division working in conjunction with the traffic engineer, because highway traffic is naturally the most important consideration in connection with street and highway planning. Regulations designed to best serve the traffic on city and rural highways are very often simply a temporary expedient best suited for existing conditions, knowing that the proper solution can only be brought about through changes in the existing street or highway system. This is pretty generally the case, particularly in the older sections of built-up communities as the street systems were not laid out for present day traffic. So the present traffic regulations and the proposed changes in the street systems to improve conditions are so closely inter-related that their functions should be closely allied if not included in the same division.

There can be no solution of the traffic problem without it being the result in a very large measure, of comprehensive planning for the future, which includes provisions for belt line highways, grade separations, widening and extension of street systems, underground sidewalks, bridge approaches, arcading, consideration of zoning provisions, etc. The problem, therefore, is primarily one of planning in general, of which highways and street systems constitute the backbone.

The enforcement, however, of traffic regulations is naturally a proper function of the police. There should, of course, be the closest cooperation between the traffic and planning divisions, and the enforcement division.

HIGHWAY GRADE CROSSING SEPARATIONS

Highway grade separations are most important, and have not been considered seriously enough in the past. Very often separating the grades where two main highways intersect will more than double the traffic capacity of the highways.

Cook County, Illinois. About a year ago, the Cook County, Illinois, Commissioners appointed a Cook County Grade Crossing Advisory Committee, and a program of highway grade crossing separations has been recommended involving 30 highway grade separations and 8 combined highway and railway grade separations. The commission made the statement that the intersections of important highways are the determining factors governing the rapidity of the movement of the traffic on those highways.

Bronx and Westchester County. An outstanding illustration of a highway system without grade crossings is that of the Bronx and Westchester County Park Commissions of New York. This highway system, when finished, will consist of 163½ miles of parkway. Thirty-six miles of these are now completed and 20 miles additional are under construction, the greater part of which will be completed by the end of 1931. This highway system is without grade crossings. As a matter of fact, with so many heavy traffic highways crossing, it would be practically impossible to carry the traffic that these highways are called upon to accommodate if the grades of intersecting highways were not separated.

New Jersey. Studies were made in New Jersey to determine upon the economics of proposed highway grade separations, and the published reports show that the separations greatly increase the traffic capacity. Grade separations will often make it unnecessary to widen existing highways or build parallel routes to accommodate the traffic and for a far less cost. New Jersey has already provided for several grade separations and has planned for several more where heavy trffic routes intersect.

Philadelphia Tri-State District. There are many instances in the Philadelphia Tri-State District where grade separations at intersections of very heavily traveled highways are the only solution of the problem. The importance of grade separations cannot be too forcibly emphasized as they are not only essential from the standpoint of increasing the traffic capacity of the highways, but will also result in greatly decreasing the highway accidents.

PARKING

Consideration should be given to the elimination of parking on through state highways in peak hours and on Sundays and holidays. This will increase the traffic capacity over 25 per cent at least.

TRAFFIC LANE MARKING

The Report states: "On rural roads it is probable that lane marking will be valuable at intersections or in semi-congested conditions. In general, however, the committee believes it is unnecessary to lane mark four lane highways, and that a slight amount of traffic control and supervision will keep drivers in their proper positions."

It seems to me that traffic lane marking is very important for two reasons. Experience has shown that it will increase the traffic capacity of highways, and it will reduce the number of accidents. Generally, traffic lanes should be marked at intersections, vertical and horizontal curves, and at sufficient intervals on long tangents, both on two and four lane traffic highways. A vigorous campaign should be carried on to eliminate the middle of the road driver, as he is one of the principal causes of accidents. A severe penalty calculated to prevent this nuisance should be imposed.

UNIFORM SIGNALS AND SIGNS

The need of uniform signals and signs for night as well as day driving should be stressed. Night driving particularly has never been given the consideration it should. Inadequate signals not only delay traffic but cause accidents.

CITY TRAFFIC PROBLEM

The report has gone extensively into this phase of the traffic problem, has made a number of recommendations and cited a number of examples of traffic conditions. There is no cure-all for this problem. The general regulations should be uniform throughout the country, so that strangers will know what to expect, but the details concerning the operation and regulation of the traffic in each city are a problem unto themselves. They depend on the social and economic conditions, the street lay-outs, the location of business and residental sections, and so forth.

SIDEWALKS

One consideration to which the report does not call attention is the predominance of pedestrian traffic at a number of the important intersections in the business districts. In a survey recently made in Philadelphia by the Mitten Management, it was shown that at certain downtown important intersections, 34 per cent of the people crossed in vehicles and 66 per cent on foot. Of the 34 per cent crossing in vehicles, 2 per cent were in taxis, 24 per cent in private automobiles and 74 per cent in trolley cars. The problem in the central business district in Philadelphia is largely pedestrian. Underground sidewalks are suggested as a possible solution.

ORIGIN AND DESTINATION TRAFFIC SURVEY

MADE BY THE REGIONAL PLANNING FEDERATION OF THE PHILADELPHIA TRI-STATE DISTRICT

THE PHILADELPHIA TRI-STATE REGION

The region covers an area of 4,000 square miles, and comprises a group of political sub-divisions, having a population of 3,500,000, so related geographically, physically, economically and socially that they have a common interest in the solution of the inter-related facilities upon which their common welfare depends. The Region embraces parts of three states, sixteen counties and more than 360 political subdivisions. It is bounded roughly by New Hope, Pa., and Princeton, N. J., on the north, Wilmington, Del., on the south, Coatesville, Pa., and Pottstown, Pa., on the west, and the New Jersey Pinelands on the east.

The Regional Planning Federation of the Philadelphia Tri-State District, in cooperation with the chief engineering officials of the Region and the Technical Advisory Committee, which consists of the outstanding engineers, architects and landscape architects of the Region, is engaged in preparing a comprehensive plan to provide for the orderly development of all the major physical facilities necessary to the best interests of the social and economic life of the people living in the interdependent communities that compose the region.

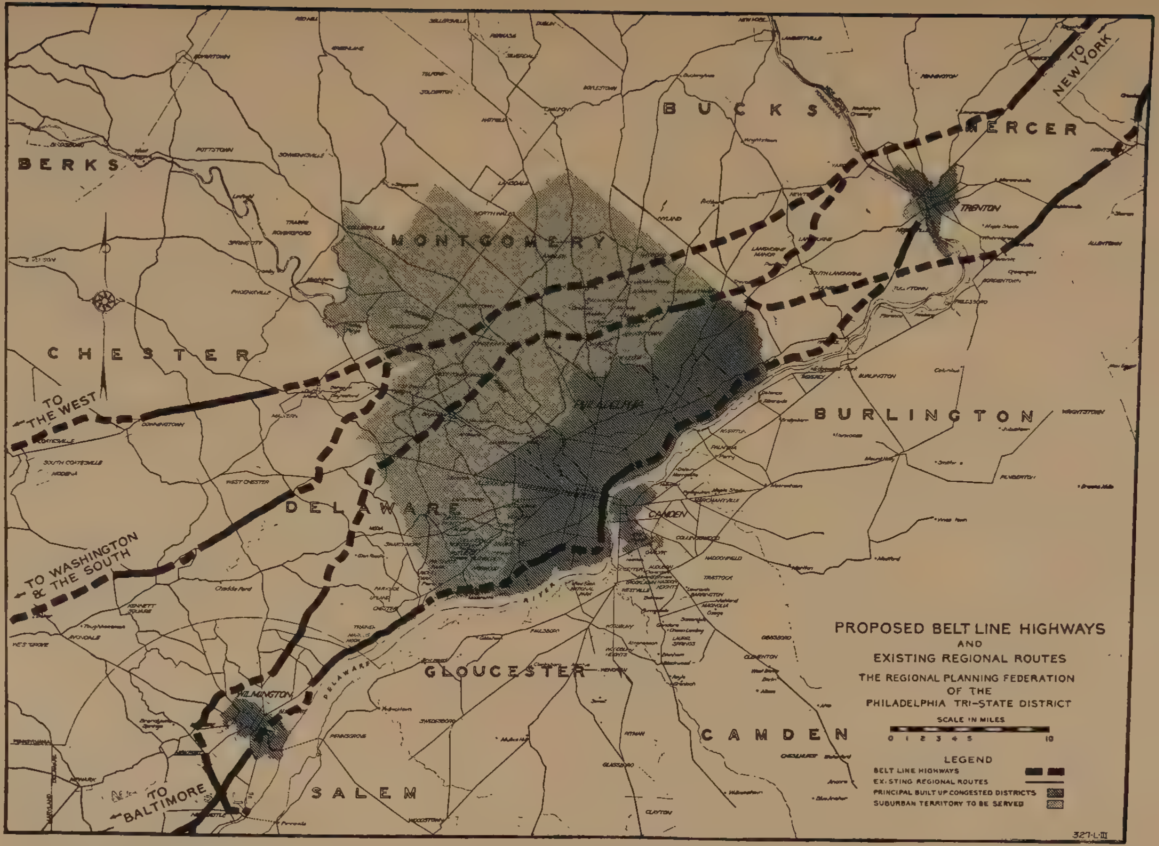
The major physical facilities include highways, railways, airways, ports and waterways; parks, parkways and recreational centers; use of land; and sanitation, drainage and water supply.

An ideal Regional Plan in a more or less built-up and thickly populated area can be accomplished only through the cooperative effort of all its political subdivisions. Regional planning is an effort not only to arrest, but to correct so far as possible, haphazard developments resulting from failure to visualize the future needs.

The highways are, of course, the backbone of a Regional Plan.

THE SURVEY AND SOME OF THE RESULTS

The task of preparing a comprehensive plan for the development of the Philadelphia Tri-State District requires a very intimate knowledge of the movement of people in and through this vast region. Basic to any such planning, of course, is the study of existing transportation facilities, population and its distribution and rate of growth, and knowledge concerning the origin and destination of the daily moving


PROPOSED BELT LINE HIGHWAYS
AND
EXISTING REGIONAL ROUTES
THE REGIONAL PLANNING FEDERATION
OF THE
PHILADELPHIA TRI-STATE DISTRICT
SCALE IN MILES
0 1 2 3 4 5 10
LEGEND
BELT LINE HIGHWAYS
EXISTING REGIONAL ROUTES
PRINCIPAL BUILT UP CONGESTED DISTRICTS
SUBURBAN TERRITORY TO BE SERVED
327-L-III
TO NEW YORK
TO THE WEST
TO WASHINGTON & THE SOUTH
TO BALTIMORE
BERKS
BUCKS
MERCER
MONTGOMERY
CHESTER
DELAWARE
BURLINGTON
GLOUCESTER
CAMDEN
SALEM
TRENTON
PHILADELPHIA
CAMDEN
WILMINGTON
DELAWARE RIVER

Proposed Routes for Belt Line Highways Around Philadelphia, Trenton and Wilmington to Carry Through Traffic as Well as to Serve Local Suburban Territory Without Going Through Congested Districts.

population. It is necessary to know the facts regarding the causes, the means and the diversity of this movement if the plans that are being formulated are to be founded on something more than the general knowledge and familiarity on the part of individuals with this movement.

Accordingly on Thursday, June 6, 1929, the Regional Planning Federation undertook a comprehensive regional Origin and Destination Traffic Survey of all modes of transportation of regional importance in the Tri-State District in cooperation with the railroads, bus companies, inter-urban electric lines and the highway departments of the three states concerned. Close to a million questionnaire cards were distributed to the traveling public and about half of this number were given to persons using the highways. That phase of the survey concerned with highway traffic is presented herewith.

The purpose of the survey was to obtain not only a relative idea of the "from where to where" movement of the people in motor vehicles, but also to show the "how" and "why" of this movement.

The highway questionnaire cards contained nine pertinent questions, briefly as follows:

1. From where
2. To where
3. Kind of vehicle
4. Driver or passenger
5. Number in vehicle
6. Time received
7. Other transportation used
8. Time for trip
9. Purpose

The cards were designed so that a simple checking of items listed would suffice for answers in the majority of cases, and simplify the work of tabulating the results.

The survey was preceded by a very extensive publicity campaign to acquaint the public with its purpose. This was carried on through the newspapers, by the distribution of hand-bills, and samples of the cards with instructions as to their use were also widely distributed.

The Highway Survey was made possible by the cooperation of the State Highway patrol in Pennsylvania, the New Jersey and Delaware Highway Departments, the Fairmount Park Commission, the Delaware River Bridge Joint Commission and the several ferry companies. These organizations detailed their men to count the traffic and distribute the cards at the 63 locations chosen for the work. These stations were located principally at gateways to large population centers so as to obtain the movement of traffic from suburban areas into business and industrial centers. In addition a few stations were located at important regional cross roads, thus obtaining the data for through traffic as well as that entering the cities of Philadelphia, Camden, Trenton and Wilmington. The stations were distributed as follows:

36 in Pennsylvania
6 in Delaware
6 in North New Jersey
9 in South New Jersey
6 at Delaware River Crossings
in South Jersey

Cards were distributed between the hours of 6:30 A. M. and 8:30 P. M. to each occupant of every motor vehicle passing a station except where the occupant had previously received a card from another station. Only the vehicles traveling inbound towards centers of population were counted and given cards, the feeling being that a one direction story for such a large area would be practically the same as if the survey was made for traffic traveling in both directions. The hours were extended into the evening to obtain an idea of late afternoon and early evening traffic which was generally for purpose of recreation.

The survey included the origin and destination of *all* occupants of motor vehicles in order to compare properly the results from the highway survey with those obtained on rail and bus transportation units. Furthermore, it was expected that where several persons were riding in a vehicle both the origin and destination as well as the purpose for the trip might be different. To separate the story for the driver of the vehicle as distinguished from the other occupants or passengers, a place was provided on the questionnaire card for the occupant to check as to whether he was driver or passenger in the vehicle. The business reply card privilege of the Post Office Department was utilized to obtain the return of the cards distributed at the stations. The reverse side of the questionnaire card was printed with the address of the Federation and two cents was paid to the Post Office Department for each card returned.

Somewhat over 200,000 vehicles excluding busses were counted, and the returns on the highway questionnaire cards amounted to about 20 per cent.

The following are some illustrations of the data obtained from this survey; which show the need for the proposed highways shown on page 127.

The analysis indicates that, of the traffic coming from Trenton and north and west of Trenton and the New York region, crossing the Delaware into Pennsylvania on the Lincoln highway at Morrisville.

64 per cent was destined for the built-up section of Philadelphia and points between Trenton and Philadelphia.

15 per cent was destined for the immediate suburbs, north and west, of the built-up portion of Philadelphia.

7 per cent destined for Wilmington, Del., and points within the region beyond the Philadelphia suburbs.

14 per cent through traffic to more distant points to the south and west.

This analysis shows that 14 per cent may be classed as strictly through traffic to distant points, but 36 per cent would naturally use a by-pass or belt line highway beginning near Trenton and passing Philadelphia on the north and west. The Pennsylvania State Highway Department is now making surveys and studies with a view to eventually provide such a highway.

An analysis of the traffic from the west and south also indicates that about the same percentage would naturally use a belt line highway around Philadelphia toward Trenton.

The survey, and also a count made by the State Highway Department of New Jersey, indicate that about 50 per cent of the traffic from Philadelphia and points south and west, going north toward Trenton is destined to points beyond Trenton. All of this traffic would flow into the belt line highway north and west of Philadelphia, if it also by-passed Trenton to the north and connected with the direct route to the New York region. A highway to meet this situation is shown on the map and is under consideration by the Pennsylvania and New Jersey State Highway Departments. Surveys are now being made for the section of this highway which will start at the Lincoln Highway at Philadelphia City Line, cross the Delaware River north of Yardley, and connect with the proposed New Jersey State Highway which will pass Trenton to the north and connect with the New Jersey direct route to New York.

An outer belt line highway from New Jersey to Maryland, connecting with routes north and south, is also under consideration and the traffic analysis indicates that it will not be many years before this outer belt line highway will be required to relieve the inner belt line from the strictly through traffic.

The analysis also indicates, as previous traffic studies have, the necessity for the proposed industrial highway from Trenton to Chester and Wilmington, and the early construction of the section between Philadelphia and Wilmington.

An analysis of the traffic over the Philadelphia-Camden Delaware River Bridge and the ferries shows that about 76 per cent constitutes regular daily traffic between the built-up section of Philadelphia and Camden and South Jersey. This analysis is from a count made on Thursday, June 6, 1929, so it does not give the story of the shore traffic which is not well under way until later in the season, and is at its peak on Saturdays, Sundays and holidays. The summer shore holiday traffic, of course, consitutes the greatest portion of the traffic on the Delaware River Bridge. The analysis emphasizes the necessity for belt line highways around all large centers of population, connecting with radial highways to the different business and residental centers to diffuse the traffic.

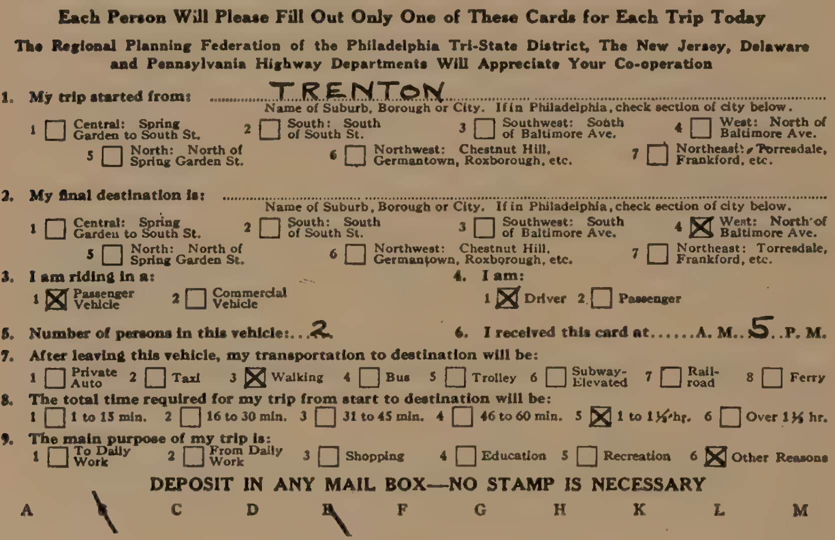
Through traffic is always considerably greater on Saturdays, Sundays and holidays, and also during the tourist season which starts in July and ends in the early fall. A count, taking this traffic into consideration, would increase the average percentage of the traffic destined to go around Philadelphia to Trenton, and likewise, around Trenton to destinations beyond.

These are just a few illustrations of the results of the survey. The Federation now has complete information relative to the origin and destination of the traffic in the region, and of the traffic passing through the region. From this information a comprehensive highway system is being planned, which will enable traffic to go to and from the points of origin and destination by the most direct routes.

PROCEDURE IN ANALYZING RETURNS FROM REGIONAL HIGHWAY ORIGIN AND DESTINATION SURVEY

John Nolen, Jr., *Regional Planning Federation of the Philadelphia Tri-State District.* The analysis of the vast amount of information obtained in the survey involved primarily a consideration of the reliability of the data and secondly provision, in tabulating, for applying it in a great variety of ways. There were 41,000 cards returned, for each of which there were, to the nine questions, a possible 150 different answers, not including a multiplicity of combinations of these answers. The task was to group, and finally tabulate this information in a simple, concise and usable manner, so as to obtain a complete inventory of returns, and so arranged that any desirable grouping of results could be extracted readily. Throughout every operation and listing, the data had to be related to the per cent return which the original cards represented at each of the 63 points from which they were distributed, for only in this way could a measure of accuracy of the known results be obtained.

An idea of the extent of the analysis may be visualized from a study of a facsimile of the card (Figure 1).



Each Person Will Please Fill Out Only One of These Cards for Each Trip Today

The Regional Planning Federation of the Philadelphia Tri-State District, The New Jersey, Delaware and Pennsylvania Highway Departments Will Appreciate Your Co-operation

1. **My trip started from:** TRENTON
   Name of Suburb, Borough or City. If in Philadelphia, check section of city below.
   1 ☐ Central: Spring Garden to South St. 2 ☐ South: South of South St. 3 ☐ Southwest: South of Baltimore Ave. 4 ☐ West: North of Baltimore Ave.
   5 ☐ North: North of Spring Garden St. 6 ☐ Northwest: Chestnut Hill, Germantown, Roxborough, etc. 7 ☐ Northeast: Torresdale, Frankford, etc.
2. **My final destination is:** ……………
   Name of Suburb, Borough or City. If in Philadelphia, check section of city below.
   1 ☐ Central: Spring Garden to South St. 2 ☐ South: South of South St. 3 ☐ Southwest: South of Baltimore Ave. 4 ☒ West: North of Baltimore Ave.
   5 ☐ North: North of Spring Garden St. 6 ☐ Northwest: Chestnut Hill, Germantown, Roxborough, etc. 7 ☐ Northeast: Torresdale, Frankford, etc.
3. **I am riding in a:**
   1 ☒ Passenger Vehicle 2 ☐ Commercial Vehicle
4. **I am:**
   1 ☒ Driver 2 ☐ Passenger
5. **Number of persons in this vehicle:** 2
6. **I received this card at** ……A. M. 5 P. M.
7. **After leaving this vehicle, my transportation to destination will be:**
   1 ☐ Private Auto 2 ☐ Taxi 3 ☒ Walking 4 ☐ Bus 5 ☐ Trolley 6 ☐ Subway-Elevated 7 ☐ Rail-road 8 ☐ Ferry
8. **The total time required for my trip from start to destination will be:**
   1 ☐ 1 to 15 min. 2 ☐ 16 to 30 min. 3 ☐ 31 to 45 min. 4 ☐ 46 to 60 min. 5 ☒ 1 to 1½ hr. 6 ☐ Over 1½ hr.
9. **The main purpose of my trip is:**
   1 ☐ To Daily Work 2 ☐ From Daily Work 3 ☐ Shopping 4 ☐ Education 5 ☐ Recreation 6 ☒ Other Reasons

DEPOSIT IN ANY MAIL BOX—NO STAMP IS NECESSARY

A B C D E F G H K L M

Figure 1.

It will be noted that the card was designed so that a simple checking of items listed would suffice for answers in the majority of cases, and would simplify the work of tabulating the results.

Identification of each card with the station at which it was distributed was made by coding the cards before distribution to the various stations. The letters along the bottom of the card were checked with crayon by warping a handful of cards and marking the letters indicating each station.

This provision made it possible to determine the route of travel between origin and destination and to compute a per cent return for each sation, by comparison of the cards returned with the volume count. If the results of the tabulation of data from each of the 63 stations were to be comparable, one with another, adjustment must be made for the variation in per cent return at different stations. It would not be permissible to combine or compare the origins and destinations indicated at a station for which there was a 10 per cent return with those at a station for which there was say a 20 or 25 per cent return, unless allowances for these variations were first made. Furthermore, by multiplying the cards returned by a factor based on the per cent return, it would be possible to obtain a quantitative estimate of traffic between origin and destination.

Although over 200,000 vehicles, excluding buses, were counted on the day of survey, calculation has shown there were approximately 170,000 *different* vehicles contacted carrying somewhat over 300,000 persons. Due to the fact that some stations were so situated as to cause many vehicles to be counted more than once, the actual per cent return could only be obtained after allowances for this duplication were made. This net return after corrections averaged over 17 per cent, varying generally between 10 and 30 per cent at the different stations. From the occupants of the vehicles there were a total of 41,000 usable cards, 29,000 being from the drivers. There was an average of 1.77 persons per vehicle.

In order to analyze the origin and destination data of the survey, the region was divided into zones for grouping the origins and destinations indicated on the cards. Thirty-three zones were chosen within the region and in order to provide for the listing of traffic coming into or going out of the region, 14 additional outside zones were chosen, which were generally directional in character, but which enclosed important areas like the New York Region, Seashore Resorts and the Baltimore-Washington area. This made a total of 47 in all. The boundaries of these zones coincided with county lines or subdivisions of them, and enclosed areas of similar function, having a common community of interest. By adopting boundaries along political subdivision lines, comparisons can be made between the traffic generated, the population and motor vehicle registration. This should make possible the establishment of a law of traffic relationship between zones which can be expressed in terms of population, registration, area and distance.

These zones were next assigned code numbers and all the cards coded for the proper zone of origin and destination. Individually each card was then checked for consistency of the information shown upon it, and adjustments made where obviously items had been incorrectly checked, or were not according to standard.

For the tabulation of the results, the data on the original survey cards were then transferred, by adoption of code numbers, to Hollerith tabulating machine cards. The process of sorting and tabulating this varied mass of information was thus mechanized and shortened, for the Hollerith machines sort cards at the rate of 400 a minute

The tabulated data were entered upon forms, 11 by 17 inches in size, designed specially for the purpose. Three major tabulations were made and information taken off in the following sequence:

I. Station.
    A. Time Handed Out.
        1. Occupant (i. e. Driver or Passenger).
            a. Kind of Vehicle.
                i. Number in Vehicle (Drivers' Cards Only).
II. Origin Zone.
    A. Destination Zone.
        1. Station.
            a. Occupant.
                i. Kind of Vehicle.
                ii. Purpose.
III. Station.
    A. Destination Zone.
        1. Occupant.
            a. Facility to Destination.

After tabulation, it was necessary to prorate a small percentage of the minor data that were unchecked on the original cards. This was done in accordance with ratios shown for the checked items. Additional adjustments were also made, principally for trips indicated in the " reverse direction by error," than that in which the survey was taken.

Out of a possible 47 by 47, or 2209 origin and destination combinations, there were 1302 for which the survey showed movement of people. Out of these there were only 277 or 21 per cent for which the returns indicated only one person moving between zones. The complexity and magnitude of the regional movement can thus be imagined.

The practical application of the results of the survey depends on what is to be proved. General tendencies can be worked out from the tabulated results of the actual cards returned, but for a true comparison of traffic between zones, the final adjustment for the variation in per cent return at different stations is necessary. This then provides a basis for establishing quantitative estimates of the origin-destination composition of traffic at any point. It is to be noted that, while the number of returns indicating trips between certain zones may be small, the outstanding fact is that a combination of many small and different trips together establishes a large and definite relationship at any one point between the groups of zones on either side. These relationships can then be expressed most forcibly in terms of percentage.

With the data analyzed and tabulated as described above, they can be expended to almost any degree, or adapted to a variety of uses. When the analysis is finally completed the survey will have supplied in addition to origin and destination relationships, basic data on the purposes for which motor vehicles are used, and the time spent in making trips, the relation between home and work, and finally the importance of coordinating all other transportation facilities with the highway system.

While the work of analyzing the results from such a survey has a diversity of difficulties, perhaps the most important consideration is to organize most carefully and completely the task of actually making the survey. Properly executed, the results will be exceedingly worth while, and lend themselves readily to systematic analysis.

Mr. V. R. Burton, *Deputy Highway Commissioner, Michigan.* I was very much impressed in the uniformity of the traffic which Dean Johnson has shown on the charts. They show a condition comparable to that which we have in Michigan in our heavy industrialized areas, but not at all typical of what we have in our agricultural and resort regions. In the resort regions variation is very great. They have a very sparse population and a very high number of vehicles during July and August. During the winter time, because of the small population—only the main roads being open—the traffic may amount to only 20 to 25 per cent of the average yearly traffic. During July and August that traffic commonly runs from 300 to 400 per cent of the average yearly traffic. The average traffic as determined in Michigan is made up of traffic counts taken in four traffic quarters and these traffic quarters were determined by the flow of funds from the gas taxes. We know exactly what the medium month in each of the traffic quarters is and the counts are taken in that medium month. A good many of the traffic counts taken have been very misleading because they did not represent the average throughout the entire year, and where the traffic is so variable, if you want an accurate average it is necessary to carry on the traffic count throughout the year. That is one of the reasons why our general traffic averages show very much less traffic than some taken in a number of other states whose counts extend only through the summer season. In the industrial areas our counts show very much the same variation that Dean Johnson's do. In the agricultural areas there is not as wide a difference as there is in the resort areas. This is something with which we must concern ourselves very much in the design of our highways. We feel that the order of improvement should be determined by the average traffic and that the design must be determined by the peak traffic, and if we are to determine the proper order of improvement and adequate design, it is very necessary that our traffic counts be taken in a great deal of detail and studied with much care.

Mr. Carl F. Meyer, *Worcester Polytechnic Institute.* I would like to say something about signals, especially signalizing of intersections, and with particular reference to Massachusetts. Traffic control is a very pressing matter in this state. We are forty-fifth among the 48 states in mileage of state highways, twelfth in volume of registration, and first in the number of cars per mile of highway. Massachusetts has just made a pioneer attempt at a uniform system of signalizing intersections.

By legislative action electric traffic control signals in Massachusetts either in cities or towns or on state and county highways in rural districts can only be erected on state highways by written approval of the Department of Public Works, and only at intersections and only used during the time of day, when traffic volume warrants. There have been many signals erected by counties and towns without regard to uniformity and often without regard to traffic volume. Signals have not served to expedite traffic. They have often retarded it. The department specifies the minimum traffic requirements for signalizing an intersection, these figures being a total of at least 500 vehicles per hour with at least 125 per hour on the cross streets. Even though the total traffic is considerably in excess of 500 vehicles per hour, the signal is not considered warranted unless the cross street has a traffic of at least 125. These are minimum requirements and are not to be interpreted as necessitating signal control. However, if the pedestrian traffic on the cross street is of sufficient volume the signal may be allowed even though vehicular traffic volume is much less than 125 per hour.

The department has made a survey of the types of signals in use in the state, and has attempted to specify the type and the location of the signal which according to the present state of our knowledge, will conform with good engineering principles, at the least expense for changing the existing signals. The principal features in addition to the traffic requirements have to do with location and type. All signals installed in the state from now on, as the result of definite traffic counts, must be of the three lens vertical type with four signals, one at each of the far right hand corners. The locations of the lenses are also definitely fixed. The red lens must be at the top with the yellow and green in that order, vertically. The lenses must be separately illuminated and with a bulb of at least 60 watts to furnish the necessary illumination. The clear height from the road to the signal is a function of the distance between the stop line on the near side of the intersection and the signal on the far side, and that varies from eight to sixteen feet in the specifications. No turns of any kind are allowed when the red light is shown unless a green arrow lens pointing left or right is illuminated at the same time. This lens is allowed only when the streets are wide enough to allow separate lanes for such turning traffic. It is recommended that no attempt be made to synchronize signals at intersections spaced more than 200 feet apart.

I have just jotted down these points, and I am sure that any of the contact men or others who wish to acquaint themselves with the details of this system can secure Traffic Bulletin No. 2 by writing the Department of Public Works in Boston Massachusetts.

Professor R. L. Morrison, *University of Michigan.* As was brought out this morning, all of us know that, in general, the same people pay for building the roads and operating them. If the value of vehicle time is assumed to be $3.00 per hour, and if every one of our motor vehicles, 25,000,000 of them, is delayed three minutes a day due to congestion from lack of grade separations, or other causes, the total cost per year is nearly $1,400,000,000.

Professor W. E. Lay, *University of Michigan.* I would like to see some kind of regulation of the green and red advertising signs. They are often about the same height as the traffic control signals, and contribute greatly the difficulties for the average driver.

Chairman Mattimore. We gave that suggestion on the law, in Pennsylvania especially on the gas station that have the red blinkers.

THE THEORY OF EXTERNAL LOADS ON CLOSED CONDUITS IN THE LIGHT OF THE LATEST EXPERIMENTS

ANSON MARSTON, *Director*
Iowa Engineering Experiment Station

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I. Scope of Application and Definitions

Working over a period of about twenty-one years, and assisted at different times by several persons,* the author of this paper has developed, and from time to time published,† in successive stages, a complete mathematical theory of external loads on closed conduits and of the supporting strengths of pipe conduits. This theory applies to all kinds of closed conduits and to all classes of field conditions of conduit construction.

The theory is supported strongly, and with remarkable unanimity: First, by all actual weighings on record of loads on closed conduits determined in experimental researches; second, by extensive field data obtained by several series of widespread examinations of actual conduits in use.

The term " conduits " includes culverts, drains, sewers, aqueducts, water pipes, gas mains, telephone conduits and underground steam mains; of all shapes, materials, degrees of rigidity, field construction conditions affecting loads, and field construction conditions affecting supporting strengths.

As to shapes, closed conduits may be rectangular, arched, oval, circular, or of any other shape.

As to materials, closed conduits may be of masonry (brick, plain or reinforced concrete, or stone) constructed in place or otherwise: or of *pipes or conduit sections,* of burnt clay, or plain or reinforced concrete, for culverts, sewers, tile drains, or telephone or other conduits; or of *metal,* including cast iron, steel or wrought iron and corrugated metal; or of *wood,* including box sections and wood stave pipe, and bored wood pipes.

As to degrees of rigidity, closed conduits may be classified as:

1. *Rigid conduits,* whose cross sectional shapes cannot be distorted sufficiently to change their vertical or horizontal dimensions more than 0.1 per cent without causing materially injurious cracks; including all rectangular conduits, and all cylindrical conduits made of plain or reinforced concrete masonry or pipes, or of burnt clay pipes.
2. *Semi-rigid conduits,* whose cross sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions

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* See Acknowledgments, Appendix A.

† See List of Publications of the Theory of External Loads on Closed Conduits, Appendix B.

more than 0.1 per cent, but not more than 3.0 per cent, without causing materially injurious cracks; including segmental block conduits, and those made of cast iron pipe, together with some brick or stone block masonry cylindrical conduits.

3. *Flexible conduits,* whose cross sectional shapes can be distorted sufficiently to change their vertical or horizontal dimensions more than 3.0 per cent before causing materially injurious cracks; including those made of corrugated pipe, thin steel or wrought iron pipe, and probably some cylindrical conduits made of brick or stone block masonry.

As to field construction conditions affecting external loads, closed conduits may be classified as:

1. *Ditch conduits,* completely buried in ditches excavated in comparatively solid and inert materials.
2. *Projecting conduits,* under embankments but projecting above the embankment sub-grades.

The field construction conditions affecting external loads on projecting conduits may be classified as:

1. *The ditch condition,* a typical case of which is a very flexible culvert projecting above the sub-grade under a low embankment.
2. *The incomplete ditch condition,* a typical case of which is a very flexible culvert projecting above the sub-grade under an embankment of considerable height.
3. *The incomplete projection condition,* a typical case of which is a rigid or semi-rigid culvert projecting above the sub-grade under an embankment of considerable height.
4. *The projection condition,* a typical case of which is a rigid or semi-rigid culvert projecting above the sub-grade under a low embankment.

Note.—There are some minor modifications of the above four main classes.

Field construction conditions affecting supporting strengths of closed conduits may be classified as follows:

1. *Impermissible bedding,* is that method of bedding conduits in which materially insufficient care is exercised to shape the foundation surface to fit the lower part of the conduit exterior and to refill all spaces under and around the conduit with granular materials at least partially compacted.
2. *Ordinary bedding,* is that method of bedding conduits completely buried in ditches in which the conduit is bedded, with "ordi-

nary" care, in an earth foundation shaped to fit the lower part of the conduit exterior with reasonable closeness for a width of at least 50 per cent of the conduit breadth; and in which the remainder of the conduit is surrounded to a height of at least 0.5 feet above its top by granular materials, shovel placed and shovel tamped to completely fill all spaces under and adjacent to the conduit; all under the general direction of a competent engineer.

2a. *Ordinary projection bedding,* is that method of bedding projecting conduits under embankments in which the conduit is bedded, with "ordinary" care in an earth foundation shaped to fit the lower part of the conduit exterior with reasonable closeness for a width of at least 50 per cent of the conduit breadth; and in which the remainder of the conduit is surrounded by granular materials, shovel placed and shovel tamped to completely fill all spaces under and adjacent to the conduit; all under the general direction of a competent engineer.

3. *First class bedding,* is that method of bedding conduits completely buried in ditches in which the conduit is carefully bedded on fine granular materials in an earth foundation carefully shaped to fit the lower part of the conduit exterior for a width of at least 60 per cent of the conduit breadth; and in which the remainder of the conduit is entirely surrounded to a height of at least 1.0 foot above its top by granular materials carefully placed by hand to completely fill all spaces under and adjacent to the conduit, and thoroughly tamped on each side and under the conduit so far as practicable, in layers not exceeding 0.5 feet in thickness; all under the direction of a competent engineer, represented by a competent inspector constantly present during the operation.

3a. *First class projection bedding,* not yet defined.

4. *Concrete-cradle bedding,* is that method of bedding conduits in which the lower part of the conduit exterior is bedded in plain or reinforced concrete, of suitable thickness under the lowest part of the conduit exterior and extending upward each side of the conduit for a greater or less proportion of its height.

II. EXTERNAL LOADS ON CLOSED CONDUITS

The most important external loads on closed conduits result from the action of gravity. Their components parallel to the conduit axis are so relatively unimportant that, for practical purposes, the load at any point on the exterior surface is sufficiently expressed by its vertical and horizontal components acting in the perpendicular cross sectional plane at the point. The vertical components of such loads are usually much greater and more important than the horizontal components, and the latter are most conveniently expressed in terms of their ratios to the vertical components.

Hence, the mathematical formulas for computing external loads on closed conduits most conveniently express the vertical load components only, leaving the horizontal pressures to be calculated from their ratios to the vertical loads.

The external loads on closed conduits comprise: First, the downward loads, applied usually to the upper portions of the conduit exteriors; second, the resultant upward foundation pressures, applied usually to the lower portions of the conduit exteriors. The total foundation pressure upward is equal to the total load downward.

The external loads on closed conduits are of two classes:

First, the *loads due to the filling* materials placed over and around the conduits.

Second, *the loads transmitted through the filling materials but due to extraneous super-loads,* applied at the upper surface of the fill or at some lower level higher than the top of the conduit.

Super-loads may be concentrated as in the case of truck wheel loads, or they may be distributed, as in the case of piles of construction materials at the fill surface.

III. The Mathematical Theory of Loads on Closed Conduits Due to Fill Materials

A. NOTATION

Let B = horizontal breadth of a conduit or ditch, feet
B_c = greatest horizontal breadth of a conduit, feet
B_d = horizontal breadth of a ditch at top of conduit, feet
C = conduit load calculation coefficient, abstract number
C_c = load calculation coefficient for conduits projecting above the adjacent embankment sub-grade, abstract number
C_d = load calculation coefficient for conduits completely buried in ditches, abstract number
H = vertical height from top of conduit to the upper surface of the fill, feet

H_e = "height of equal settlement," = vertical height from the top of the conduit to the level at and above which the fill materials directly over the conduit settle equally with the adjacent fill materials, feet

p = the projection ratio, = the ratio of the *vertical height of the top of the conduit above the embankment sub-grade level* to B_c, abstract number

NOTE.—The conduit projection $= pB_c$.

d_c = *the increment of deflection of the conduit* (*i. e.*, the shortening of its vertical height), feet

s_f = *the increment of settlement of the conduit foundation,* feet

s_g = *the increment of settlement of the embankment sub-grade,* feet

s_m = *the increment of settlement of materials adjacent to the conduit between the levels of the conduit top and the adjacent embankment sub-grade,* feet

NOTE.—d_c, s_f, s_g and s_m are the increments due to the addition at or above the height of equal settlement of any (the same for all) incremental, uniform layer of fill materials.

r_{sd} = the "settlement deflection ratio," abstract number

$$r_{sd} = \frac{(s_m + s_g) - (d_c + s_f)}{s_m}$$

NOTE.—r_{sd} is *negative* for the "complete" and the "incomplete ditch condition."

r_{sd} is *positive* for the "complete" and the "incomplete projection condition."

W_c = the vertical external load on a closed conduit due to fill materials, pounds per foot length

w = the unit weight of fill materials, pounds per cubic foot

K = the ratio of *active horizontal pressure* at any point in the fill to the *vertical pressure which causes the active horizontal pressure,* abstract number

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} \text{ (Rankine's formula)}$$

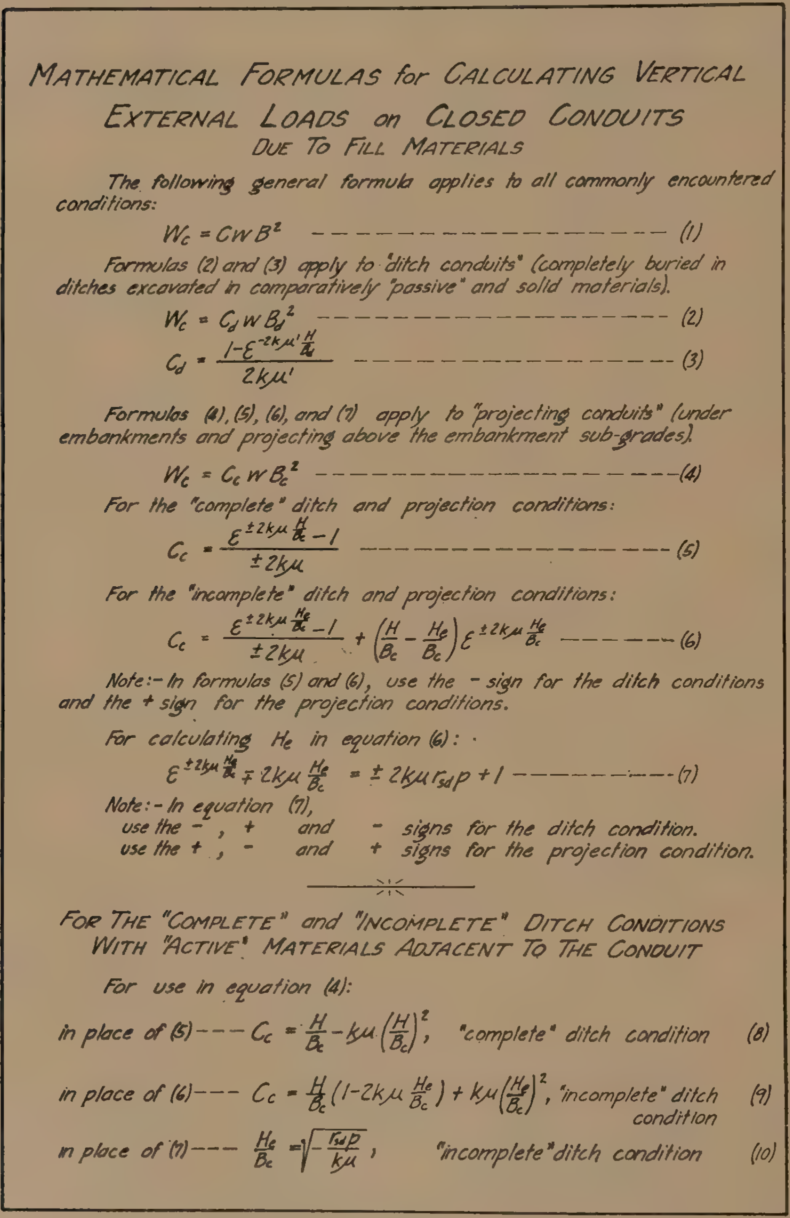
μ = the "*coefficient of internal friction*" in the fill materials, abstract number

μ' = the "*coefficient of sliding friction*" between the fill materials and the sides of the ditch, abstract number

NOTE.—μ' may be less than μ, but cannot be greater.

ϵ = 2.7182818 = base of natural logarithms, abstract number

B. MATHEMATICAL FORMULAS FOR CALCULATING VERTICAL EXTERNAL LOADS ON CLOSED CONDUITS



MATHEMATICAL FORMULAS for CALCULATING VERTICAL EXTERNAL LOADS on CLOSED CONDUITS

DUE TO FILL MATERIALS

The following general formula applies to all commonly encountered conditions:

$$W_c = CwB^2 \text{ ----------------- } (1)$$

Formulas (2) and (3) apply to "ditch conduits" (completely buried in ditches excavated in comparatively "passive" and solid materials).

$$W_c = C_d w B_d^2 \text{ ----------------- } (2)$$

$$C_d = \frac{1-\varepsilon^{-2k\mu'\frac{H}{B_d}}}{2k\mu'} \text{ ----------------- } (3)$$

Formulas (4), (5), (6), and (7) apply to "projecting conduits" (under embankments and projecting above the embankment sub-grades).

$$W_c = C_c w B_c^2 \text{ ----------------- } (4)$$

For the "complete" ditch and projection conditions:

$$C_c = \frac{\varepsilon^{\pm 2k\mu\frac{H}{B_c}}-1}{\pm 2k\mu} \text{ ----------------- } (5)$$

For the "incomplete" ditch and projection conditions:

$$C_c = \frac{\varepsilon^{\pm 2k\mu\frac{H_e}{B_c}}-1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)\varepsilon^{\pm 2k\mu\frac{H_e}{B_c}} \text{ -------- } (6)$$

Note:- In formulas (5) and (6), use the − sign for the ditch conditions and the + sign for the projection conditions.

For calculating H_e in equation (6):

$$\varepsilon^{\pm 2k\mu\frac{H_e}{B_c}} \mp 2k\mu\frac{H_e}{B_c} = \pm 2k\mu r_{sd} p + 1 \text{ ------------ } (7)$$

Note:- In equation (7),
use the − , + and − signs for the ditch condition.
use the + , − and + signs for the projection condition.

* * *

FOR THE "COMPLETE" and "INCOMPLETE" DITCH CONDITIONS WITH "ACTIVE" MATERIALS ADJACENT TO THE CONDUIT

For use in equation (4):

in place of (5) --- $C_c = \frac{H}{B_c} - k\mu\left(\frac{H}{B_c}\right)^2$, "complete" ditch condition (8)

in place of (6) --- $C_c = \frac{H}{B_c}\left(1-2k\mu\frac{H_e}{B_c}\right) + k\mu\left(\frac{H_e}{B_c}\right)^2$, "incomplete" ditch condition (9)

in place of (7) --- $\frac{H_e}{B_c} = \sqrt{-\frac{r_{sd}p}{k\mu}}$, "incomplete" ditch condition (10)

Figure 1

C. COMPUTATION DIAGRAMS FOR EXTERNAL VERTICAL LOADS ON CLOSED CONDUITS DUE TO FILL MATERIALS

The computation of vertical external loads on closed conduits by formulas (2) and (4), (Figure 1), is made very simple and easy by computation diagrams, from which the values of the calculation coefficient C_d and C_c can be obtained directly instead of by special computations.

For "Ditch Conduits" Completely Buried in Ditches

$$W_c = C_d w B_d^2 \tag{2}$$

The values of C_d may be read directly on Figure 2, herewith, reproduced from Bulletin 47, Iowa Engineering Experiment Station, without special computations by equation (3).

For "Projecting Conduits" (Under Embankments and Projecting Above the Embankment Sub-Grade)

$$W_c = C_c w B c^2 \tag{4}$$

The values of C_c may be read directly on Figure 3, herewith, without special computation by equations (5), (6), (7), or (8), (9), (10), (Figure 1).

D. PHYSICAL FACTORS REQUIRED FOR COMPUTATIONS OF EXTERNAL LOADS ON CLOSED CONDUITS, DUE TO FILL MATERIALS

Three special physical factors required in using equations (1) to (10), (Figure 1), are:

1. The coefficient, μ, of internal friction in the fill material.
2. The projection, pB_c, of the conduit above the adjacent embankment sub-grade.
3. The settlement-deflection ratio, r_{sd}, of the *relative* settlements of the embankment sub-grade, the fill materials adjacent to the conduit, and the conduit foundation, together with the deflection of the conduit top.

1. The Coefficient of Internal Friction, μ, in the Fill Material

The values of μ for use in the formula should be those corresponding to the pulls required to establish slow *continued* motion in the usual laboratory tests (in which bottomless boxes of the material are pulled over horizontal surfaces of the same material).

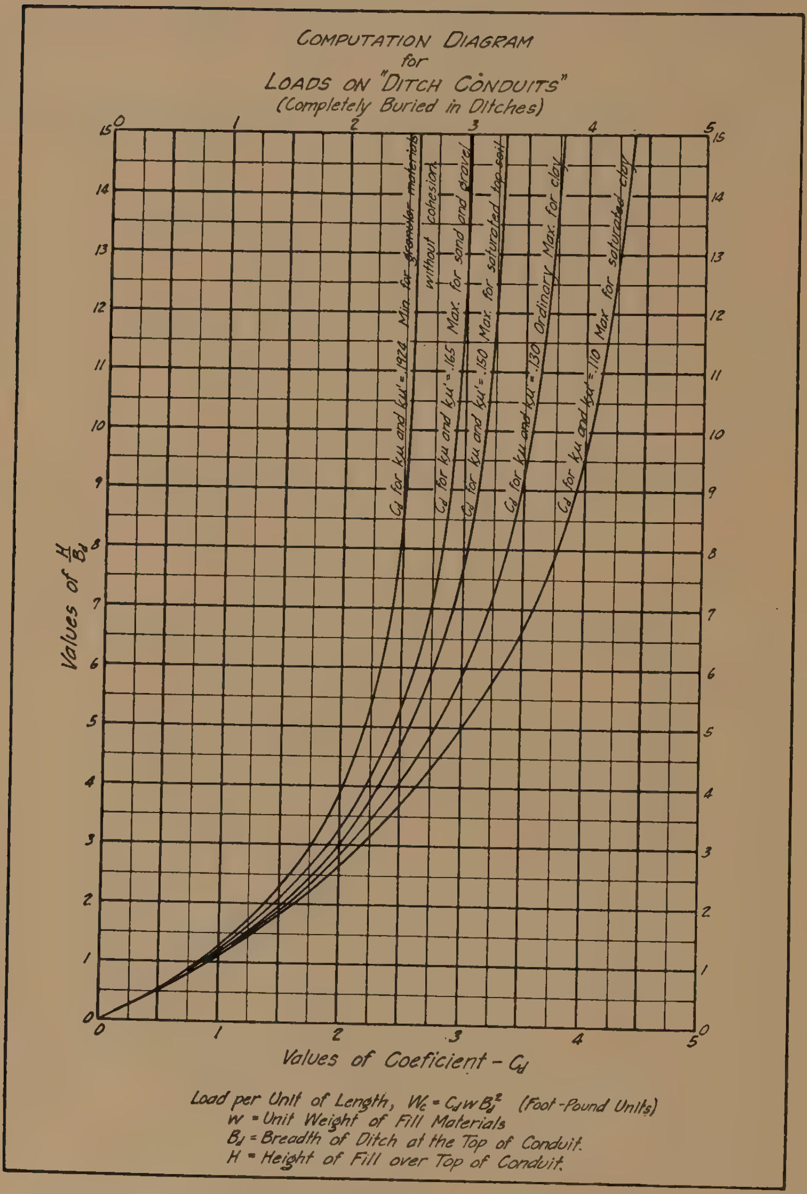

COMPUTATION DIAGRAM
for
LOADS ON "DITCH CONDUITS"
(Completely Buried in Ditches)
C_d for ku and ku' = .1924 Min. for granular materials without cohesion
C_d for ku and ku' = .165 Max. for sand and gravel
C_d for ku and ku' = .150 Max. for saturated top soil
C_d for ku and ku' = .130 Ordinary Max. for clay
C_d for ku and ku' = .110 Max. for saturated clay
0
1
2
3
4
5
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Values of $\frac{H}{B_d}$
Values of Coeficient – C_d
Load per Unit of Length, $W_c = C_d w B_d^2$ (Foot-Pound Units)
w = Unit Weight of Fill Materials
B_d = Breadth of Ditch at the Top of Conduit.
H = Height of Fill over Top of Conduit.

Figure 2

That these are the correct values to use has been clearly proven: First, by hundreds of comparisons of computed loads with actual loads on conduits, as determined by actual weighings in experiments; second, by extensive field data of actual conduits whose supporting strengths are known by laboratory tests.

The values of μ obtained as above are somewhat of the nature of the semi-empiric physical factors, or coefficients, so common in engineering problems.

No laboratory methods for measuring μ have yet been devised in which the pressure conditions in actual fills are reproduced with near approximation. In the actual fill, there is sufficient time for important slight movements of the grains of material into positions of comparatively stable equilibrium, and they apparently are held in comparatively stable equilibrium against rolling along the vertical plane of slipping by the comparatively large vertical pressures which cause the smaller horizontal pressures which cause the friction. In the laboratory tests (in which the plane of slipping is usually rotated 90°), the time adjustment of the grains in the actual fill is mainly absent, the major (vertical) pressures of the actual fill are entirely omitted, the grains along the plane of slipping are in comparatively unstable equilibrium against rolling, and an initial slipping occurs at a fraction of the true internal friction pull. This phenomenon has long been known, and was reannounced by the author in Bulletin 31, Iowa Engineering Experiment Station, 1913.

NOTE.—The above discussion applies also to μ', the coefficient of sliding friction.

2. The Projection, pB_c, of the Conduit Above the Adjacent Embankment Sub-Grade

The projection, pB_c, has an important effect (see Figure 3) on loads on "projecting conduits" (under embankments and projecting above the embankment sub-grade), though it does not materially affect loads on "ditch conduits" (completely buried in ditches).

In many cases, the ratio p should be regarded as a semi-empiric physical factor, representing the best judgment of the engineer as to the characteristic field conditions in actual use. Most conduits under embankments are buried partially, or even entirely in ditches which vary as to depths, widths and side slopes.

Whenever practicable, however, p should be determined by actual measurement, or observation, or from correct field construction notes, for each structure.

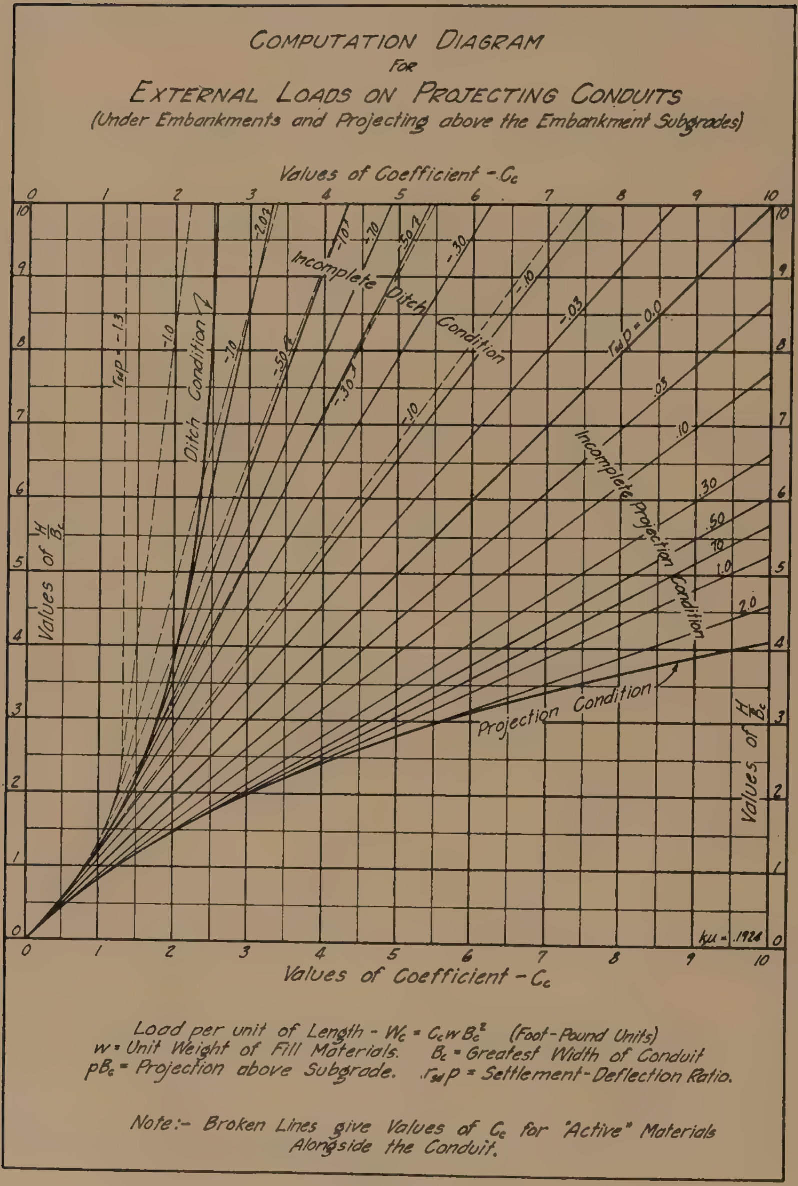

COMPUTATION DIAGRAM
FOR
EXTERNAL LOADS ON PROJECTING CONDUITS
(Under Embankments and Projecting above the Embankment Subgrades)
Values of Coefficient - Cc
0
1
2
3
4
5
6
7
8
9
10
Values of H/Bc
Ditch Condition
Incomplete Ditch Condition
Incomplete Projection Condition
Projection Condition
rsdp = -1.3
-1.0
-.70
-.50
-.30
-.10
-2.03
-1.01
-.70
-.50
-.30
-.10
-.03
rsdp = 0.0
.03
.10
.30
.50
.70
1.0
2.0
kμ = .1924
Load per unit of Length - Wc = Cc w Bc² (Foot-Pound Units)
w = Unit Weight of Fill Materials. Bc = Greatest Width of Conduit
pBc = Projection above Subgrade. rsdp = Settlement-Deflection Ratio.
Note:- Broken Lines give Values of Cc for "Active" Materials Alongside the Conduit.

Figure 3

3. *The Settlement-Deflection Ratio,* r_{sd}

The settlement-deflection ratio, r_{sd}, is an extremely important factor affecting loads on "projecting conduits" (under embankments and projecting above the embankment sub-grade). It indicates correctly the effects of the conduit deflections, the settlement of the conduit foundations and the embankment sub-grade settlements, relative to the yielding of the embankment fill materials adjacent to the conduits. Loads on conduits under embankments of the same height are sometimes three times as great for high values of r_{sd} as for low values. See Figure 3.

The settlement-deflection ratio is a semi-empiric physical factor, the values of which to use in computations must be determined by comparisons of computed loads on conduits with the actual loads, as weighed in experiments and as estimated for actual conduits by their supporting strengths as determined by laboratory tests.

A table of values of r_{sd} for different field and conduit conditions is under preparation by the Iowa Engineering Experiment Station.

E. THE HORIZONTAL COMPONENTS OF EXTERNAL LOADS ON CLOSED CONDUITS DUE TO FILL MATERIALS

Study of hundreds of measurements of actual horizontal pressures in granular fill materials, during 21 years of research, has convinced the author of the reliability of Rankine's well-known formulas, which may be stated as follows:

The *Active* horizontal pressure $= K \times$ (vertical pressure).

The *Passive* horizontal pressure $\ngtr 1/K \times$ (vertical pressure).

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu}$$

1. *For "Ditch Conduits" (Completely Buried in Ditches)*

The horizontal pressures against "ditch conduits" are wholly dependent on the class of bedding, and are best taken into account by using *conduit supporting strength bedding ratios* (see page 168), of field supporting strength to laboratory test strength.

2. *For "Projecting Conduits" (Under Embankments and Projecting Above the Embankment Sub-Grades)*

(a) For *rigid conduits,* not injuriously cracked:

Active horizontal pressure equal to $K \times$ (vertical pressure) should be assumed against the projecting conduit sides.

(b) For *semi-rigid conduits:*

Somewhat greater active horizontal pressures should be assumed than for rigid conduits.

(c) For *flexible conduits,* and for all badly cracked conduits:

The development of passive resistance to elongation of the horizontal diameters will often cause horizontal pressures equal to the vertical pressures.

NOTE.—The Iowa Engineering Experiment Station is endeavoring to establish, experimentally, "moduli of passive resistance," per inch elongation of horizontal diameter, for different materials and classes of conduits.

IV. THE MATHEMATICAL THEORY OF LOADS ON CLOSED CONDUITS DUE TO SUPER-LOADS

A. NOTATION

Let A = length of a section of a closed conduit, feet

a = area of each elementary area into which the conduit top is divided for computation purposes, square feet

> NOTE.—The vertical loads on the tops of cylindrical and arched conduit tops, due to super-loads, are substantially equal to the vertical loads, due to super-loads, on the projection of the conduit section upward on the horizontal plane tangent to the conduit top. Hence, the elementary areas, *a,* may be taken in this "*conduit top horizontal plane projection.*"

B_c = the greatest horizontal breadth of the conduit, feet

B_d = the horizontal breadth of a ditch, at top of conduit, feet

C_t = calculation coefficient for vertical loads on conduit tops due to concentrated super-loads, abstract number

C_{us} = calculation coefficient for vertical loads on the top of "ditch conduits" (completely buried in ditches), due to uniformly distributed super-loads, abstract number

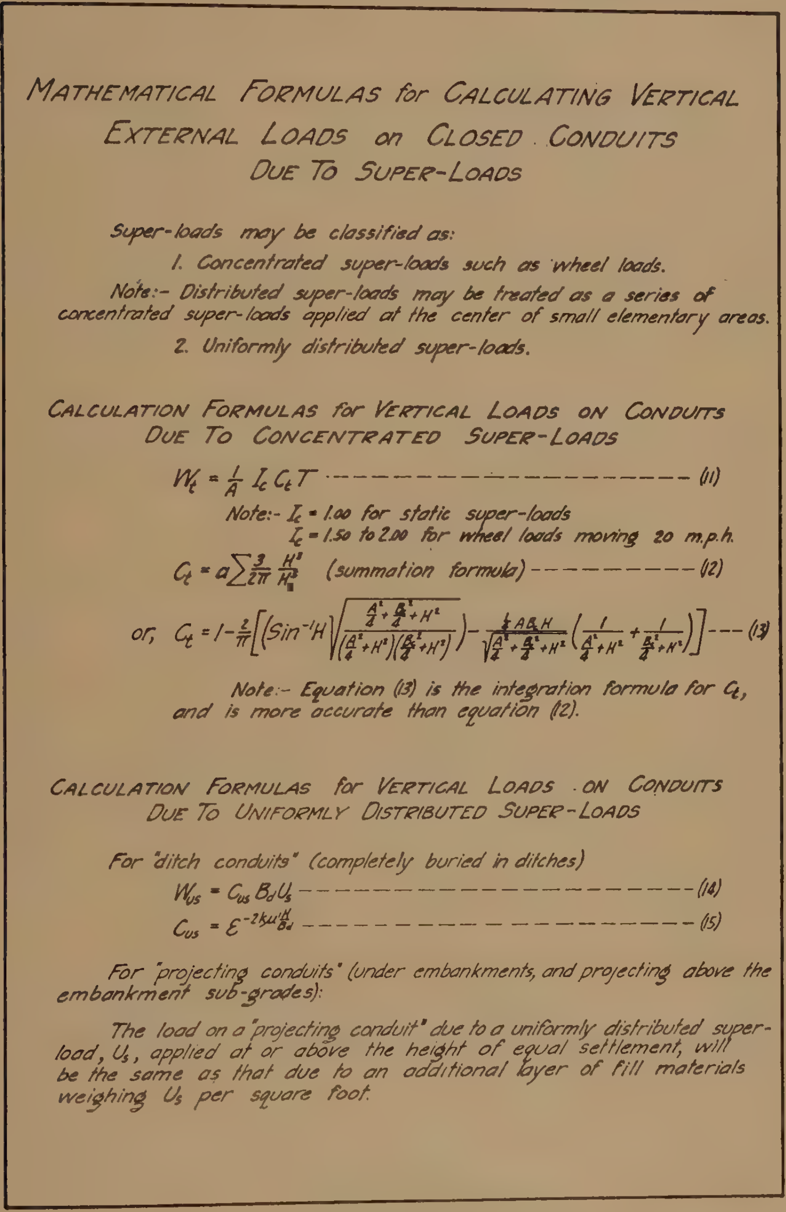
H = the *vertical height* from the level of the conduit top to the level of the point or area of application of a super-load, feet

H_s = the *slant height* from any point x, y in the conduit top horizontal plane projection to the point of application of a concentrated super-load, feet

x and y = the rectangular coordinates of any point x, y in the conduit top horizontal plane projection, with x parallel and y perpendicular to the conduit axis, feet

I_c = impact coefficient for the calculation of impact loads on conduit tops, due to moving concentrated super-loads, abstract number

B. MATHEMATICAL FORMULAS FOR CALCULATING VERTICAL EXTERNAL LOADS ON CLOSED CONDUITS DUE TO SUPER-LOADS



MATHEMATICAL FORMULAS for CALCULATING VERTICAL EXTERNAL LOADS on CLOSED CONDUITS DUE TO SUPER-LOADS

Super-loads may be classified as:

1. Concentrated super-loads such as wheel loads.

Note:- Distributed super-loads may be treated as a series of concentrated super-loads applied at the center of small elementary areas.

2. Uniformly distributed super-loads.

CALCULATION FORMULAS for VERTICAL LOADS ON CONDUITS DUE TO CONCENTRATED SUPER-LOADS

$$W_t = \frac{1}{A} I_c C_t T \text{ -------------------- } (11)$$

Note:- $I_c = 1.00$ for static super-loads

$I_c = 1.50$ to 2.00 for wheel loads moving 20 m.p.h.

$$C_t = a \sum \frac{3}{2\pi} \frac{H^3}{H^5} \text{ (summation formula) ---------- } (12)$$

$$\text{or, } C_t = 1 - \frac{2}{\pi}\left[\left(\operatorname{Sin}^{-1} H \sqrt{\frac{\frac{A^2}{4} + \frac{B^2}{4} + H^2}{\left(\frac{A^2}{4} + H^2\right)\left(\frac{B^2}{4} + H^2\right)}}\right) - \frac{\frac{1}{2} ABH}{\sqrt{\frac{A^2}{4} + \frac{B^2}{4} + H^2}}\left(\frac{1}{\frac{A^2}{4} + H^2} + \frac{1}{\frac{B^2}{4} + H^2}\right)\right] \text{ --- } (13)$$

Note:- Equation (13) is the integration formula for C_t, and is more accurate than equation (12).

CALCULATION FORMULAS for VERTICAL LOADS ON CONDUITS DUE TO UNIFORMLY DISTRIBUTED SUPER-LOADS

For "ditch conduits" (completely buried in ditches)

$$W_{us} = C_{us} B_d U_s \text{ ---------------------- } (14)$$

$$C_{us} = \varepsilon^{-2k\mu' \frac{H}{B_d}} \text{ ---------------------- } (15)$$

For "projecting conduits" (under embankments, and projecting above the embankment sub-grades):

The load on a "projecting conduit" due to a uniformly distributed super-load, U_s, applied at or above the height of equal settlement, will be the same as that due to an additional layer of fill materials weighing U_s per square foot.

Figure 4

$T =$ a concentrated super-load, pounds

$U_s =$ a uniformly distributed super-load, pounds per square foot

$W_{us} =$ the *average* total vertical load on a section of a closed conduit, due to a uniformly distributed super-load U_s, pounds per linear foot

$W_t =$ the *average* total vertical load on a section of a closed conduit, due to a concentrated super-load T, pounds per linear foot

$K =$ ratio of active horizontal pressure at any point in the fill material to the *causal* vertical pressure, abstract number

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} \text{(Rankine's formula)}$$

$\mu =$ *coefficient of internal friction* of the fill material, abstract number

$\mu' =$ *coefficient of sliding friction* between the fill material and the sides of a ditch, abstract number

$\pi = 3.14159 =$ the ratio of a circumference of a circle to its diameter, abstract number

C. COMPUTATION DIAGRAM AND TABLE FOR EXTERNAL VERTICAL LOADS ON CLOSED CONDUITS, DUE TO SUPER-LOADS

The computation of vertical external loads on closed conduits by formulas (11) and (14) (Figure 4) is made very simple and easy by a computation diagram, for formula (11), and a computation table for formula (14), from which the values of the calculation coefficients C_t and C_{us} can be obtained directly instead of by special computations.

For Concentrated Super-Loads

$$W_t = 1/A I_c C_t T \tag{11}$$

The values of C_t may read directly from computation diagrams, of which Figure 5 is a sample.

For Loads on Conduits in Ditches, Due to Uniform Super-Loads U_s

$$W_{us} = C_{us} B_d U_s \tag{14}$$

The values of C_{us} may be taken directly from Table 1, herewith, reproduced from Bulletin 31, Iowa Engineering Experiment Station, 1913.

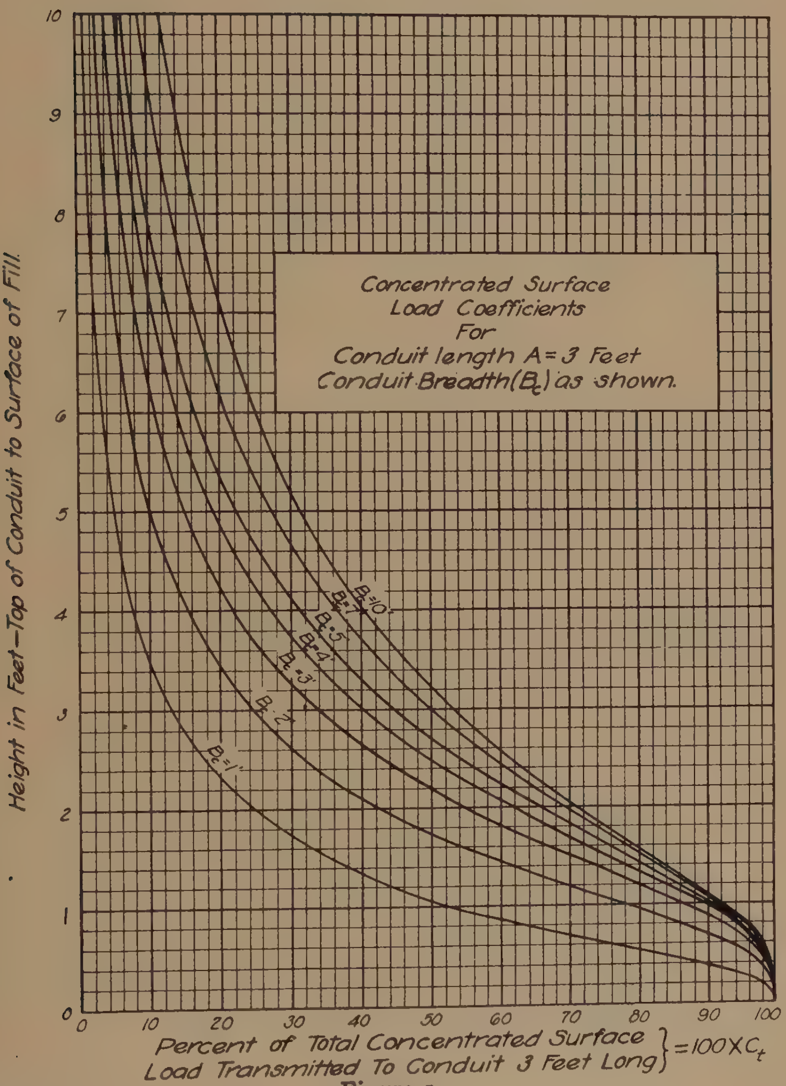

Concentrated Surface
Load Coefficients
For
Conduit length A = 3 Feet
Conduit Breadth (B_c) as shown.
B_c=10'
B_c=7'
B_c=5'
B_c=4'
B_c=3'
B_c=2'
B_c=1'
Height in Feet—Top of Conduit to Surface of Fill.
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60
70
80
90
100
Percent of Total Concentrated Surface Load Transmitted To Conduit 3 Feet Long } = $100 \times C_t$

Figure 5

TABLE I

Safe Values of C_{us} to Use in Formula (14)

H/B_d	Sand and damp top soil	Saturated top soil	Damp yellow clay	Saturated yellow clay	H/B_d
0.0	1.00	1.00	1.00	1.00	0.0
0.5	0.85	0.86	0.88	0.89	0.5
1.0	0.72	0.75	0.77	0.80	1.0
1.5	0.61	0.64	0.67	0.72	1.5
2.0	0.52	0.55	0.59	0.64	2.0
2.5	0.44	0.48	0.52	0.57	2.5
3.0	0.37	0.41	0.45	0.51	3.0
4.0	0.27	0.31	0.35	0.41	4.0
5.0	0.19	0.23	0.27	0.33	5.0
6.0	0.14	0.17	0.20	0.26	6.0
8.0	0.07	0.09	0.12	0.17	8.0
10.0	0.04	0.05	0.07	0.11	10.0

V. The Characteristics of Loads Due to or Transmitted by Granular Fill Materials

The mathematical theory of external loads on closed conduits, as presented on pages 142 to 154, applies to loads due to or transmitted by granular fill materials. Such loads have certain characteristics which must be clearly understood in applying the mathematical theory correctly.

1. Loads due to or transmitted by granular fill materials vary greatly with variations in the properties of the materials; as to weight, settlement, moisture, temperature, internal friction and cohesion.
2. Formulas (1) to (15), of the mathematical theory of external loads on closed conduits, with proper selection of the physical factors μ, p and r_{sd}, give the *ultimate limiting loads,* to which *any* particular conduit *may* be subjected eventually; and which, on the other hand, any particular conduit *may* escape for a long time, sometimes until its removal for other causes than load failure. See Figure 6, reproduced from Bulletin 79, Iowa Engineering Experiment Station, 1926. (Cooperative with the U. S. Bureau of Public Roads).
3. The external loads on closed conduits vary greatly with time. In 7 experiments at Ames, Iowa, daily observations over periods of 3, 7½, 8, 24, 24, 24, and 28 months, respectively, have proven that loads on conduits measured soon after the completion of the fill over them should be increased 20 to 25 per cent to allow safely for the *ultimate* loads which would be

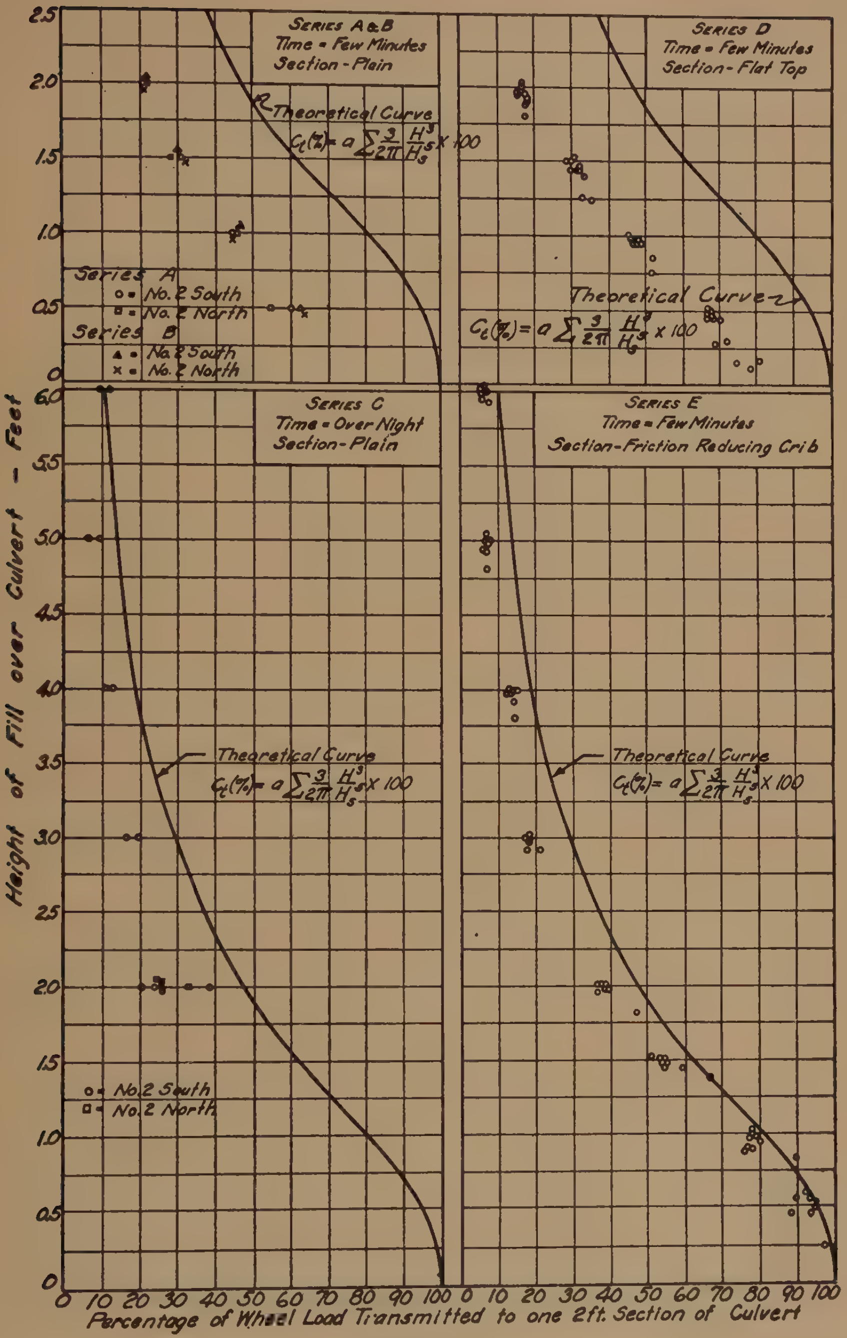

Series A & B
Time = Few Minutes
Section - Plain
Theoretical Curve
$C_t(\%) = a\sum\frac{3}{2\pi}\frac{H^3}{H_s^5} \times 100$
Series A
○ = No. 2 South
□ = No. 2 North
Series B
▲ = No. 2 South
× = No. 2 North
Series D
Time = Few Minutes
Section - Flat Top
Theoretical Curve
$C_t(\%) = a\sum\frac{3}{2\pi}\frac{H^3}{H_s^5} \times 100$
Series C
Time = Over Night
Section - Plain
Theoretical Curve
$C_t(\%) = a\sum\frac{3}{2\pi}\frac{H^3}{H_s^5} \times 100$
○ = No. 2 South
□ = No. 2 North
Series E
Time = Few Minutes
Section - Friction Reducing Crib
Theoretical Curve
$C_t(\%) = a\sum\frac{3}{2\pi}\frac{H^3}{H_s^5} \times 100$
Height of Fill over Culvert - Feet
2.5
2.0
1.5
1.0
0.5
0
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0 10 20 30 40 50 60 70 80 90 100
0 10 20 30 40 50 60 70 80 90 100
Percentage of Wheel Load Transmitted to one 2 ft. Section of Culvert

Figure 6

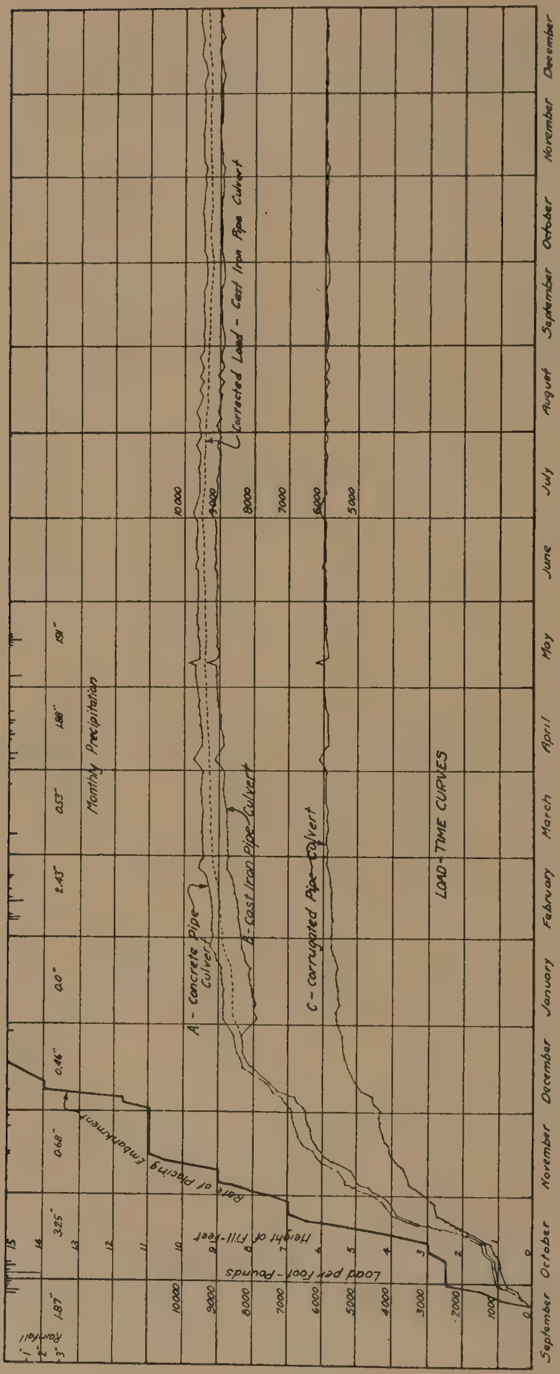


Figure 7. Experiments at Ames, Iowa, with Three Culverts of 49" Outside Diameter

found if the observations were extended to cover a period of several years. See Figure 7, reproduced from a forthcoming bulletin of the Iowa Engineering Experiment Station (Cooperative with the U. S. Bureau of Public Roads).

Field Observations of cracked conduits show that such cracks often develop months or even years after construction.

A. THE HEIGHT OF EQUAL SETTLEMENT IN EMBANKMENTS OVER CLOSED CONDUITS

The key to the mathematical solution of the theory of loads on "projecting conduits" (under embankments and projecting above the adjacent embankment sub-grade), is the discovery and publication by the author, in 1922, of the existence and the method of computation of the "height of equal settlement," at and above which, in the "incomplete" ditch and projection conditions, all additional uniform horizontal layers of embankment materials settle equally in and on each side of the vertical prism of fill materials directly over the conduit.

Below the height of equal settlement, the horizontal layers of fill material each side of the vertical prism over the conduit, settle, less in the "incomplete ditch condition" and more in the "incomplete projection condition," than the fill materials originally at the same level in the prism over the conduit.

The existence of the "height of equal settlement" was discovered and first announced by the author on purely mathematical grounds. There are many difficulties in the way of measuring the actual settlements at different points in the interiors of embankment masses, which, however, have been overcome by very ingenious apparatus devised by the staff of the Iowa Engineering Experiment Station. The actual existence of the height of equal settlement is now amply verified by many actual measurements, made during a period of years, of the actual settlements at different points in embankment interiors.

Such measurements are illustrated by Figure 8, reproduced from a forthcoming bulletin of the Iowa Engineering Experiment Station (Cooperative with the U. S. Bureau of Public Roads).

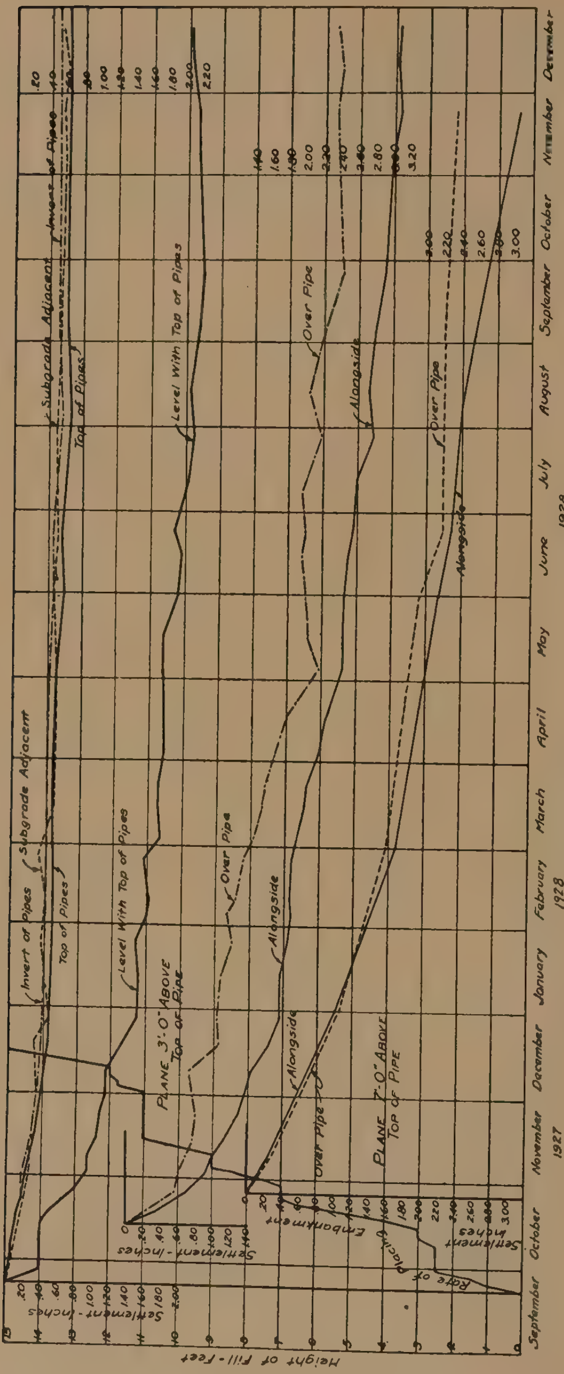


Figure 8. Time-Settlement Curves. Concrete Pipe Culverts

VI. Verification of the Mathematical Theory of External Loads on Closed Conduits, by Experiments and by Field Observations

The mathematical theory of external loads on closed conduits, as presented in equations (1) to (15) (Figures 1, 4), has been verified in the amplest manner:

> First, by actually weighing the loads on full size conduit sections, in experiments which doubtless have cost more than $150,000, at Ames, Iowa, and elsewhere, and which at Ames, Iowa, have extended continuously for 18 years and are still in progress.
>
> Second, by extensive and expensive failures and partial failures, of conduits in service, by injurious cracking and sometimes by collapse. Many such failures have been investigated, and they have been found to verify the mathematical theory. Important changes in drainage, sewerage and culvert engineering practice are now developing to make safe practice conform to this theory.
>
> Third, by extensive field examinations of actual conduits in service, some sound and others cracked. Most cracked conduits do not collapse, but many cases have been found of objectionable deformation, and a number of cases of collapse.

A. EXPERIMENTAL AND FIELD VERIFICATION OF THE THEORY OF EXTERNAL LOADS ON CLOSED CONDUITS IN DITCHES (SEE EQUATIONS (2) AND (3) (FIGURE 1)

The theory of external loads on "ditch conduits" (completely buried in ditches) has been verified in the amplest manner by the experiments and other researches of the Iowa Engineering Experiment Station at Ames, Iowa, from 1908 to 1929, still continuing. The theory is also verified by Barbour's experiments at Boston, Mass., in 1897.*

The theory of external loads on "ditch conduits" has also been verified in the amplest manner by actual experience with sewers, drains and culverts in use. The best sanitary engineers are now more and more coming to require concrete cradling of pipe sewers and drains wherever the author's theory indicates danger of injurious cracking by the computed loads.

The author's theory has been republished in highly condensed form in several standard textbooks on sewerage and drainage.

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* Journal of Association of Engineering Societies, Vol. 19, Dec., 1897.

B. EXPERIMENTAL AND FIELD VERIFICATION OF THE THEORY OF EXTERNAL LOADS ON "PROJECTING CONDUITS" (UNDER EMBANKMENTS AND PROJECTING ABOVE THE ADJACENT EMBANKMENT SUB-GRADE) (SEE EQUATIONS (4) TO (15) (FIGURES 1, 4)

1. Experimental Verification

The mathematical theory of external loads on closed conduits under embankments, and projecting above the adjacent embankment subgrade, has been verified in the amplest manner by the following experiments:

> *Iowa Engineering Experiment Station, Ames, Iowa, 1917 to 1929, still continuing.* Actual weighings were made of the total loads on pipe culverts, of 9, 24, 36 and 42 inches inside diameters, under embankment fills of heights above the tops of the pipes up to 4 feet for the 9 inch, 10 feet for the 24 inch, and 15 feet to 20 feet for the 36 and 42 inch culverts. The fill materials have included top-soil, sand, and gravel, weighing 96 to over 130 pounds per cubic foot. The culverts have included thin smooth steel, corrugated metal, cast iron, clay, plain concrete, reinforced concrete, and wood staves on steel forms. Since 1925, the experiments have included measurements of the settlements at various points on the embankment sub-grades and in the interiors of the embankments, as well as of the deflections of the culvert tops and the settlements of the culvert foundations.
>
> The experiments have covered ranges of the settlement-deflection ratio, r_{sd}, from -3.0 to $+1.0$.
>
> In all the experiments, the *total* load was weighed. In all cases except the 9 inch and the 24 inch culverts, the *total* load on *each* of several sections of each culvert was weighed. Many measurements of horizontal pressures have been made.
>
> Regular weighings of the loads and other observations have been continued 3 months to 24 months after the completion of the embankment fills in the different experiments, and are still continuing for 3 culverts which have already been under observation 24 months.
>
> All the Ames culvert tests have been for approximately 90 per cent projection ($p = 0.90$).

The Iowa Engineering Experiment Station experiments verify the mathematical theory stated in this paper in the most complete manner.

American Railway Engineering Association Roadway Committee Culvert Load Tests at Farina, Ill., 1925 to 1927.* Measurements by Goldbeck-Smith pressure cells were made at different points on the circumferences of 6 corrugated pipe culverts 1-24, 4-42, and 1-48 inches diameter, under 6.0 to 34.9 feet height of embankment, with similar measurements for a 24 inch by 27 inch concrete culvert under 32.2 feet height of embankment and a 42 inch cast iron culvert under 34.2 feet height of embankment. The projection of the culverts was approximately 90 per cent ($p = 0.90$). The total loads were not weighed.

In 1928 the committee published the "theory of culvert loads" which it considered would explain its observed pressure data. The committee stated in this publication that, for "Type I" culverts (rigid and semi-rigid), "the above theory, with a few slight changes in nomenclature, has been taken from various bulletins and papers by Dean Anson Marston." For "Type II" culverts (flexible), the committee repeated, with due credit to Dean Anson Marston, his solution for the case of a culvert actually buried in a ditch. In consultation with Dean Marston, and acting on a suggestion made by him, the committee worked out and published a modification of Dean Marston's solution for the "incomplete ditch condition," to apply to the flexible culverts in their experiments.

Thus the Farina experiments verify, in a general way, the author's theory presented in this paper.

University of North Carolina Culvert Load Tests, 1924-1927. The University of North Carolina, Chapel Hill, North Carolina, has conducted valuable culvert load tests in two series.

First Series,† 1924-1926. Three tests were made with 30 inch cast iron culvert pipe sections (32 inches outside diameter), projecting 50 per cent above the embankment sub-grade. In the first test, the embankment was of sand, 20 feet high; in the second, of clay 14 feet high; and in the third of clay, 11 feet high. In the third test, at the height of 4 feet a ditch was dug over the pipe and refilled with loose material, following a plan tried at Ames, in 1920, to establish an artificial ditch condition. All embankments were protected during construction by tar-

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* See page 794, Vol. 27, 1926, Proc. A. R. E. A., and page 527, Vol. 29, 1928, Proc. A. R. E. A.

† See Public Roads, Vol. 7, No. 11, January, 1927, for full report.

> paulins from rain. The first and second embankments were removed promptly after completion. The third was continued in place from February, 1925, to May, 1926, but doubtless without real penetration of moisture through the clay cover to the ditch immediately over the pipe, for the load decreased (as transfer of load to the sides of the ditch might occur through the development of cohesion in the clay) to August, 1925, and the later increase had not yet exceeded or even reached the original load, up to the date of removal in May, 1926.
>
> *Second Series,* 1926-1927. In the second series of tests, six 30-inch culverts, with 100 per cent projection, and respectively of smooth ingot iron, corrugated pipe, steel tube, cast iron pipe, reinforced concrete pipe, solid concrete plug, were tested successively on the same load weighing apparatus, under sand embankments 12 feet high above the culvert tops, protected during construction from rainfall and removed promptly after completion. No full report of these tests has been published, but a preliminary partial report has been published in the Proceedings of the Sixth Annual Meeting of the Highway Research Board, December 2-3, 1927, and the author is informed that the complete final report is to appear immediately.
>
> In all the North Carolina culvert load tests, the total loads were weighed by reliable apparatus, and the results are of great value.
>
> However, as already explained (see p. 154), the ultimate loads on conduits under embankments are practically always at least 20 per cent to 25 per cent greater than the loads observed immediately on completion of the embankments over them, and additions of 25 per cent should be made to the North Carolina culvert load test results on this account.

The data already published are not sufficient to enable a comparison of the loads measured with those computed by the author's theory, except in the case of the " ditch condition " culvert in the first series, and the concrete plug in the second series. In both these cases the observed loads closely support the formulas of the author's theory. Moreover, the load curves of the other tests already published are of such character that it is plainly evident that they conform closely to the curves of the author's theory, with suitable values of the settlement deflection ratio, r_{sd}, treated as a semi-empiric physical factor.

2. Verification by Field Experience

The mathematical theory of external loads on closed conduits under embankments, and projecting above the adjacent-embankment subgrades is also verified amply by field experience with actual culverts.

In actual culverts many cases of injurious cracking or deformation have been encountered: a number of extensive field examinations of culverts have accumulated a mass of data on such cases. Upon comparison of the loads computed by the author's formulas with the known or probable supporting strengths of the culvert pipes used, it is found that the observed facts can readily be harmonized with the theory.

VII. The Latest Closed Conduit Load Experiments

Mr. M. G. Spangler, of the staff of the Iowa Engineering Experiment Station, is conducting tests of three culverts, of 44 inches outside diameter, 25 feet apart under the same sandy loam top soil embankment, 15 feet high above the tops of the culverts, which are respectively of reinforced concrete, cast iron and corrugated metal. Extensive observations are being made of the settlements at a number of points on the embankment sub grades and in the interior of the embankment at different levels. These culverts have already been under observation for two complete years, and the work is continuing. Most valuable results have already been secured. Figures 7, 8, 9, 10, and 11 are all from Mr. Spangler's last progress report, which is to be published soon as a cooperative bulletin of the U. S. Bureau of Public Roads and the Iowa Engineering Experiment Station.

Mr. Spangler is also continuing the cooperative investigation of the ratio of the actual supporting strength of rigid pipe culverts to their laboratory test strengths with standard 3-edge bearings. See cooperative bulletin 76 (1926) Iowa Engineering Experiment Station and U. S. Bureau of Public Roads. Seven more embankments have been erected and six removed since 1926. The latest results support a ratio of about 2.25.

Mr. W. J. Schlick, also of the Iowa Engineering Experiment Station Staff, is conducting a series of cooperative experiments, partially financed by the Clay Products Association, Chicago, Illinois, upon the loads on sewer pipes in wide ditches, with vertical and with sloping sides.

His results verify the principle announced in 1913, in Bulletin 31, Iowa Engineering Experiment Station, that the load on a sewer pipe in a ditch with sloping sides is the same as in a vertical sided ditch of width equal to the width in the sloping sided ditch at the level of the top of the pipe.

Mr. Schlick finds that:

(1) The "ditch formula" (2) (Figure 1) gives the correct loads for widths of ditch not greater than $B_d = 1\frac{1}{2}B_c$.

(2) The "projection" formula (4) (Figure 1) gives the correct loads for widths of ditch greater than $B_d = 3B_c$.

(3) For widths of ditch between $1\frac{1}{2}B_c$ and $3B_c$, the values of H and of r_{sd} affect the decision between formula (2) and formula (4), in accordance with a table which Mr. Schlick is preparing.

VIII. THE THEORY OF THE SUPPORTING STRENGTH OF PIPE CONDUITS

In designing conduits to carry safely the loads to which they will be subjected in use, it is necessary to know, in advance, both the load and the supporting strength of the conduit.

The supporting strengths of conduit pipes can be determined in advance only by laboratory tests, but the laboratory test strengths in general are not the same as the supporting strengths of the pipe in actual structures, because of the differences in the distribution of the pressures on the pipe.

The actual distributions of the *normal* pressures around the three experimental culverts in the experiments at Ames, Iowa, are shown in Figures 9, 10, and 11, herewith.

Laboratory tests should be made with,

(1) A. S. T. M. Standard "Sand Bearings"
(2) A. S. T. M. Standard "3-edge Bearings"

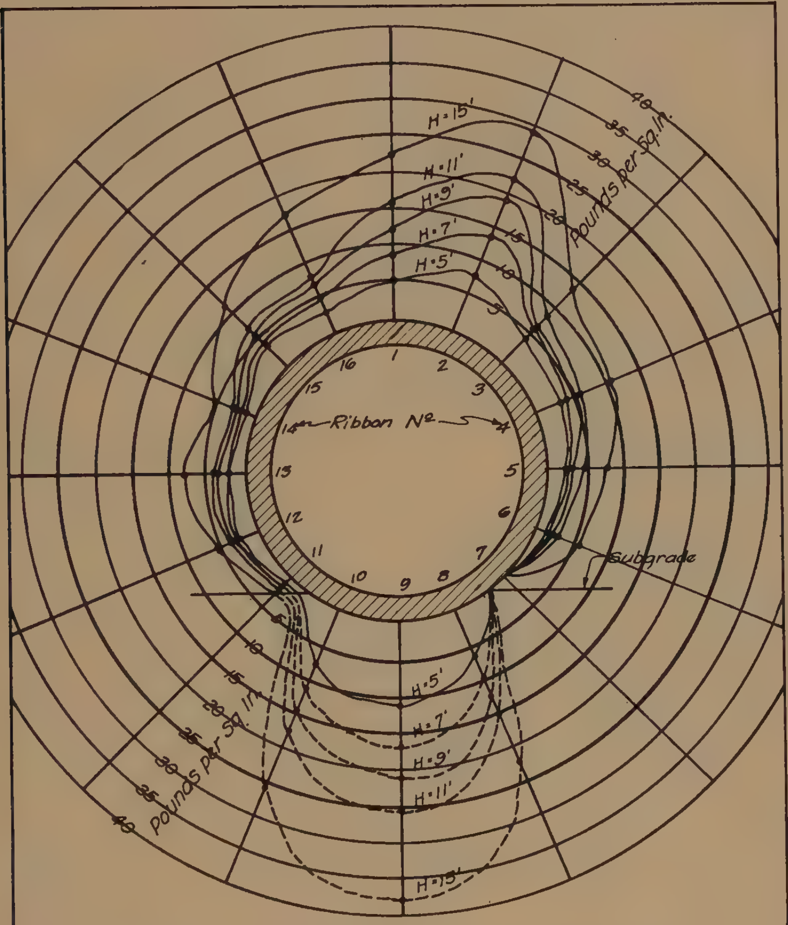
Let S_{sb} = "sand bearing" laboratory strength, pounds per foot length

S_{eb} = "3-edge bearing" laboratory strength, pounds per foot length

Notation for Laboratory and Supporting Strengths

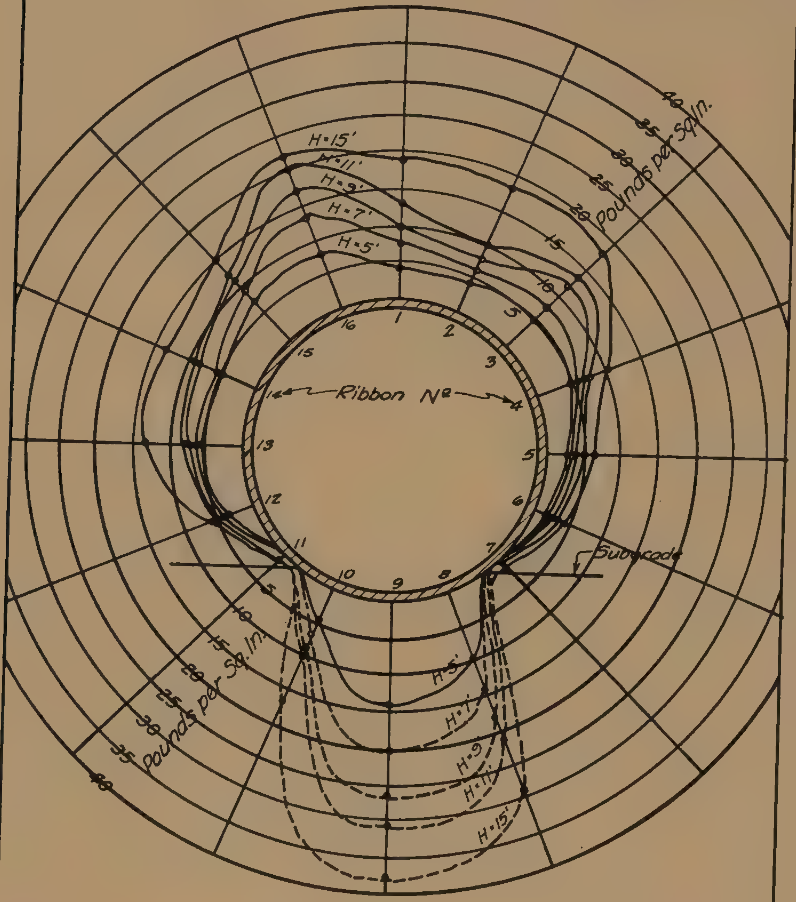
For "Ditch Conduits" (Completely Buried in Ditches)

S_{mb} = Supporting strength, "impermissible bedding," pounds per foot length



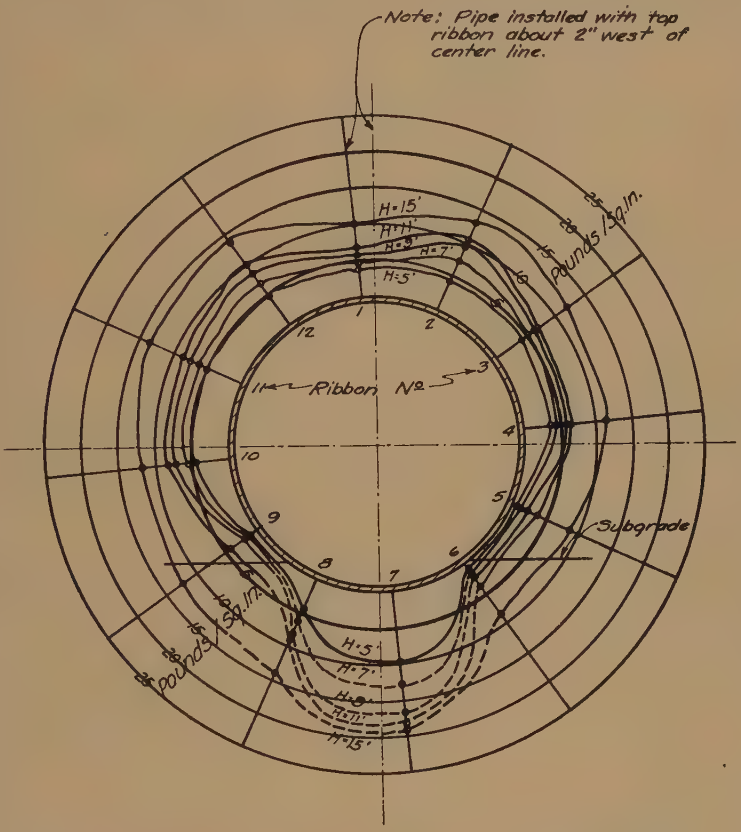
RADIAL PRESSURE ON CONCRETE PIPE CULVERT
SECTION A-3
MEASURED BY PRESSURE RIBBONS

Figure 9



RADIAL PRESSURE ON CAST IRON PIPE CULVERT
SECTION B-3
MEASURED BY PRESSURE RIBBONS

Figure 10



RADIAL PRESSURE ON CORRUGATED IRON PIPE CULVERT
SECTION C-3
MEASURED BY PRESSURE RIBBONS

Figure 11

S_{ob} = Supporting strength, "ordinary bedding," pounds per foot length

S_{cb} = Supporting strength, "1st class bedding," pounds per foot length

S_{cc} = Supporting strength, "concrete-cradle bedding," pounds per foot length.

For "Projecting Conduits" (Under Embankments and Projecting Above the Embankment Sub-Grades)

S_{opb} = Supporting strength, "ordinary projection bedding," pounds per foot length

A. THE RELATION OF ACTUAL SUPPORTING STRENGTHS TO LABORATORY TEST STRENGTHS FOR "DITCH CONDUITS" (COMPLETELY BURIED IN DITCHES)

By several years of experiments, partly cooperative with the U. S. Bureau of Public Roads, The Iowa Engineering Experiment Station has established the following relations between laboratory test strengths of conduit pipes and the actual supporting strengths of equivalent pipe used in conduits in ditches.

For "impermissible bedding," $S_{mb} = 0.75\ S_{sb} = 1.12\ S_{eb}$.

For "ordinary bedding," $S_{ob} = 1.00\ S_{sb} = 1.50\ S_{eb}$.

For "1st class bedding," $S_{cb} = 1.25\ S_{sb} = 1.87\ S_{eb}$.

For "concrete-cradle bedding," $S_{cc} = \begin{matrix}(1.50)\\(\text{to})\\(2.25)\end{matrix}\ S_{sb} = \begin{matrix}(2.25)\\(\text{to})\\(3.37)\end{matrix}\ S_{eb}$.

B. THE RELATION OF ACTUAL SUPPORTING STRENGTHS TO LABORATORY TEST STRENGTHS FOR "PROJECTING CONDUITS" (UNDER EMBANKMENTS AND PROJECTING ABOVE THE EMBANKMENT SUB-GRADE)

In several years experimentation, cooperative with the U. S. Bureau of Public Roads, the Iowa Engineering Experiment Station has determined the ratio between laboratory test strengths and actual supporting strengths, of 90 per cent projection culverts with "ordinary projection beddings," in 9 cases of 36 inch clay and concrete culverts. These seem to justify the relation (still somewhat tentative):

For "ordinary projection bedding," $S_{opb} = 1.50\ S_{sb} = 2.25\ S_{eb}$.

APPENDIX A

ACKNOWLEDGMENTS

During the past 21 years, the author has been assisted by a large number of different persons, committees, and organizations in the

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1. A. O. Anderson.
2. F. M. Okey.
3. H. W. Wagner.
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5. R. W. Crum.
6. H. F. Clemmer.
7. W. J. Schlick.
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10. J. H. Ames.
11. Bert Myers.
12. Mark Morris.
13. Robley Winfrey.
14. M. G. Spangler.
15. J. C. Everds.
16. J. W. Johnson.
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18. E. B. Smith.
19. Geo. E. Shafer, A. R. E. A. Comm.
20. D. L. Holl.
21. Comm. C-4, Am. Soc. Test. Mat.
22. Comm. C-6, Am. Soc. Test. Mat.
23. Joint Conc. Cul. Pipe Comm.
24. U. S. Bureau of Public Roads.
25. Iowa State Highway Commission.
26. Clay Products Association.

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APPENDIX B

PUBLICATION OF THE THEORY OF LOADS ON CONDUITS

The publication of the theory of loads on conduits has been made in successive stages in several papers, progress reports and bulletins, including the following:

1. Marston, Anson. Standard Tests for Drain Tile and Sewer Pipe. A. S. T. M. Proc. 11: 833-844, 1911.
2. Marston, Anson. Report on Standard Tests and Specifications for Drain Tile. (Committee C-6 Report) A. S. T. M. Proc. 13: 303-312, 1913.
3. Marston, Anson, and Anderson, A. O. The Theory of Loads on Pipes in Ditches and Tests of Cement and Clay Drain Tile and Sewer Pipe. Ia. Eng. Exp. Sta. Bul. 31, 1913.

4. Marston, Anson, Chr., and Stewart, J. T., Sec. Report of the Investigations on Drain Tile of Committee C-6 on Standard Tests and Specifications for Drain Tile. American Society for Testing Materials. Bul. 36, Ia. Eng. Exp. Sta., 1914.
5. Marston, Anson. Report on Standard Tests and Specifications for Drain Tile. (Committee C-6 Report) A. S. T. M. Proc. 14: pt. 1: 208-212, 1914.
6. Crum, R. W. Failure of a 30-inch Tile Drain at Albert Lea, Minn. A. S. T. M. Proc. 17: 453-463, 1917.
7. Marston, Anson, Schlick, W. J., and Clemmer, H. F. The Supporting Strength of Sewer Pipe in Ditches and Methods of Testing Sewer Pipe in Laboratories to Determine Their Ordinary Supporting Strength. Iowa Eng. Exp. Sta. Bul. 47, 1917.
8. Schlick, W. J. An Investigation of Tests of Iowa Shale Drain Tile. Ia. Eng. Exp. Sta. Bul. 49, 1918.
9. Schlick, W. J. Supporting Strength of Drain Tile and Sewer Pipe Under Different Pipe Laying Conditions. Ia. Eng. Exp. Sta. Bul. 57, 1920.
10. Marston, Anson. Second Progress Report to the Joint Concrete Culvert Pipe Committee. Ia. Eng. Exp. Sta. Mimeograph Report, 1922.
11. Marston, Anson. Culvert Research Report to the Advisory Board on Highway Research. Highway Research Bd. Proc. 5: pt. 1: 284-291, 1925.
12. Spangler, M. G. A Preliminary Experiment on the Supporting Strength of Culvert Pipes in an Actual Embankment. Iowa Eng. Exp. Sta. Bul. 76, 1926.
13. Spangler, M. G., Mason, Clyde, and Winfrey, Robley. Experimental Determinations of Static and Impact Loads Transmitted to Culverts. Ia. Eng. Exp. Station Bul. 79, 1926.
14. Schlick, W. J., and Johnson, J. W. Concrete Cradles for Large Pipe Conduits. Ia. Eng. Exp. Sta. Bul. 80, 1926.
15. Marston, Anson. Recent Research Relative to Culvert Pipe. (12th Annual Meeting of the American Association of State Highway Officials. Pinehurst, N. C., 1926.) Public Roads, 7: 226-229, 1927.
16. Schlick, W. J. Tests of Clay and Concrete Load Bearing Pipe A. S. T. M. Proc. 28: pt. 2: 635, 1928.
17. Schlick, W. J. Supporting Strength of Concrete-Incased Clay Pipe. Bul. 93, Ia. Eng. Exp. Station, 1929.

REPORT OF COMMITTEE ON MAINTENANCE

B. C. Tiney, *Chairman*

Maintenance Engineer, Michigan State Highway Department

The Maintenance Committee undertook during 1929 the study of two subjects, namely:

Maintenance Equipment
Bituminous Surface Treatments and Toppings

MAINTENANCE EQUIPMENT

The subject "Maintenance Equipment" may be summarized under the following headings:

1. Specifications
2. Methods of Purchase
3. History of Development
4. Rental Rates

1. Specifications

An analysis of the specifications which the various State Highway Departments furnish to bidders on equipment, indicates that an attempt is made to cover the essential features of the equipment but to leave the specifications sufficiently general to encourage competitive bidding. Further study of equipment specifications is thought desirable, with a view to developing information on the various features of each type of equipment.

2. Methods of Purchase

The committee has made a preliminary study with a view to standardizing the form in which information on equipment is submitted by bidders. Further work on this phase of equipment purchase is contemplated for the coming year. An extensive investigation of methods of purchasing equipment has been made by a joint committee of the American Association of State Highway Officials and the American Road Builders' Association, the results of which will be reported in the publications of these associations.

3. History of Development

The committee has made a study of the development of maintenance equipment. It is expected that further work will lead to improvements in design and suggestions for research.

4. Rental Rates

The determination of rental rates for state-owned maintenance equipment does not involve the same consideration as the determination of rates for privately-owned equipment used for construction purposes.

In fixing rental rates for state-owned maintenance equipment the following charges should be considered: (1) Depreciation; (2) interest on investment; (3) major or shop repairs; (4) minor or field repairs; (5) storage, incidentals, and equipment overhead; (6) insurance. Taxes would not be charged against this class of equipment as would be required witht privately-owned equipment.

In general, state-owned maintenance equipment will be used throughout its economical life whereas in the case of privately-owned equipment the necessity for replacement occasioned by frequent changes of specifications for contract jobs will result in the disposal of partly worn-out equipment at commercial prices which are almost invariably less than the economic value of such equipment. This makes for a higher rate of depreciation with a resultant increase in the amount of rental charged.

Rental rates should be sufficient to cover, during the economical life of equipment, the initial cost plus other charges as indicated above. The Associated General Contractors use a schedule of rental rates for construction equipment which has been adopted by some of the states for use in connection with maintenance equipment. It is believed that construction equipment receives harder usage than maintenance equipment and consequently should have a higher rental rate applied.

Michigan uses rental rates derived from the use of the following formula:

$$R = \frac{I + (D \times O) + (O \times M \times N),}{E}$$

and

$$C = \frac{R}{O},$$

where

R = daily rental
O = original value

N = life of equipment in years
E = No. days equipment used during life
M = per cent per year of original value necessary for maintenance (repairs and renewals)
I = interest on $\frac{1}{2}$ original value at 5 per cent for N years
D = per cent of depreciation of original value at the end of N years
C = coefficient by which original value can be multiplied to get daily rental

This formula does not take into consideration charges covering storage, incidentals, equipment overhead, and insurance. Unless accurate data has been kept for several years it becomes necessary to assign more or less arbitrary values to N, E, M and D. Coefficients have been revised from time to time when it has been found that values assigned to N, E, M or D were incorrect.

West Virginia uses the Associated General Contractors schedule of rental rates. Cost records indicate that the rates are somewhat high.

Rhode Island determines rental rates for the current year through an equalization of the average operating cost for the previous year.

Pennsylvania has established a fixed daily rental for each day the equipment is in use. This rental rate, in addition to providing for all operating and repair expenses, covers fixed charges; such as: depreciation, storage and insurance.

The U. S. Bureau of Public Roads, Division of Construction, makes a monthly charge of 4 per cent of the last inventory value of a piece of equipment. When the project has been completed or at the end of the construction season an adjustment in rentals is made as follows: The equipment is appraised and the difference between the appraised value and the value when the equipment was assigned to the job, plus the cost of needed repairs, the cost of Equipment Depot repair and replacement charges during the construction season and the cost of warehousing, shall constitute the total rental charge against the project.

California rentals are based on upkeep and replacement. Obsolescence has been found to be an important factor in determining rental rates. Some equipment is charged on a monthly basis but the majority of equipment is placed on a daily or shift basis with a minimum of ten days' rental per month regardless of use. In the establishing of rates of rental an effort is made to make each district selfsupporting; consequently if the rates used show a profit, the excess is returned to the district in which it was collected.

Necessity for Rental Rates. Depreciation, interest and the cost of maintaining equipment is obviously a charge against the project upon which the equipment is used. Any maintenance cost data which ignore the item of equipment rental do not give true cost information, and such data are useless for the purpose of comparison. California reports that 32 per cent of the total cost of maintenance is due to equipment rental. This figure checks rather closely with somewhat limited Michigan information. It is approximately correct to say that one-third of maintenance costs are chargeable to equipment rental; hence maintenance organizations that disregard rental charges are only reporting two-thirds of the actual cost.

BITUMINOUS SURFACE TREATMENTS AND TOPPINGS

The Committee Report may be summarized under the following headings:

1. Oiling of Earth Roads.
2. Use of Dust Palliative Oils on Gravel, Traffic-Bound Macadam, or other low type Metalled Surfaces.
3. Bituminous Mat Surface Treatments.
4. Bituminous Retread and other Mixed-in-Place Types.
5. Penetration Macadam and Similar Types.
6. Hot-Mixed Asphaltic Surface Courses.

1. Oiling of Earth Roads

Illinois Practice. Oiling of earth roads has been practiced extensively on county and township highways in Illinois. Reports on this work indicate that a firm subgrade and well compacted surface are necessary to successful treatment. The type of oil most favored is a medium semi-asphaltic oil, which requires heating for application. A season's treatment consists of three or four applications of $\frac{1}{4}$ gallon each, at a cost of $450.00 to $600.00 per mile 16 feet wide. These treatments are described as being satisfactory for moderate traffic, being dustless and unaffected by rains during a greater part of the season. They are not suitable where any large portion of the traffic consists of horse-drawn vehicles, farm tractors with lugs, or heavy trucks. The treatments tend to break up during the winter thaws, and the value of the oil is largely lost, necessitating, as a rule, a complete new treatment each year.

California Practice. The use of light oil as a dust layer was begun in Los Angeles County as early as 1898. It is considered to have a very definite field in the maintenance of earth roads under daily traffic of 200 to 300 vehicles. Two or three applications are made per season, at the rate of ¼ gallon per square yard for each application. Subsequent maintenance consists of blading and scarifying any portions which become rough. The cost of this work is given as $200.00 to $250.00 per mile of 18 foot road per ¼ gallon application.

Pennsylvania Practice. Oiling of earth roads is considered as being essentially a dust preventative, although the oiled surface with its characteristic of shedding water, is of value in increasing the bearing power of the road. Treatment is temporary and only effective for a few weeks. Practice is to make first application in June, and another application the latter part of August. Each application is about 0.2 of a gallon per square yard. The road surface should be well drained and dragged to acceptable cross section just prior to application of oil. Oil is applied with pressure distributor and the cost is 6.3 to 6.6 cents per gallon, applied. The annual cost of oiling earth roads varies from $300.00 to $350.00 per mile.

The U. S. Bureau of Public Roads Reports as Follows. The use of a light dust palliative oil on well compacted earth roads is indicated. The oil may be applied without heating, but application should preferably be made by means of a pressure distributor, at the rate of ⅛ to ¼ gallon per square yard. Frequency of applications to lay the dust successfully through an entire season, will depend upon traffic and climatic conditions; one application may prove sufficient, but three or four may be needed.

The Oil Processing of Earth Roads is carried on quite extensively in the West, but is successful so to speak only where the traffic is light and soil conditions favorable. Sections of road are often selected for this treatment where arid or semi-arid climate exists, or where more expensive types of surfaces are prohibitive. The basic soil should not contain any noticeable amount of coarse gravel or more than 20 per cent clay. Fine gravel or sandy soils are considered the more suitable. Satisfactory results are usually experienced where practically all of the soil material passes a ¾-inch screen, with the main portions passing a No. 10 sieve. The material passing the No. 10 sieve should range from this size to a small percentage passing a 200-mesh sieve. The roadbeds treated vary in thickness after completion from four to eight inches and carry an average of probably 7 to 8 per cent bituminous material. Fuel oil containing 60 to 70 per cent asphalt with a penetration around 80 is used in the West at times and is heated to temperature ranging from 100° to 200° F. A fuel oil higher in asphaltic content, however, is often used, as is tar and cutback material. The oil can be spread either from gravity or pressure distributors, but the latter is preferred.

2. The Use of Dust Palliative Oils on Gravel, Traffic-Bound Macadam, or Other Low Type Metalled Surfaces

Indiana Practice. Indiana has treated approximately 350 miles of gravel road with a dust palliative oil during the past season. The cost of this work is about $400.00 per mile for two treatments. The purpose is to saturate the top layer of the gravel with light oil that will hold the dust without forming a mat on the surface. On the road surface, before treatment, there should be about a one-inch layer of maintenance gravel, slag, or hard stone, which aggregate should be clean and free from clay and silt. The road surface should be smooth and free from holes before the oil is applied. The practice consists in first scraping all loose gravel to the side of the road, then applying the oil to the road, and then scraping the loose material back over the road. The first application of oil consists usually of about ½ gallon per square yard. A second very light application of ⅛ gallon per square yard may be applied as soon as dust begins to be in evidence.

The U. S. Bureau of Public Roads reports the use of dust palliative oils on macadam, shell and gravel roads. Cold application oils are applied with pressure distributors at the rate of ⅛ to ¼ gallon per square yard. From one to four applications may be necessary per season, depending on traffic and climatic conditions.

Michigan Practice. Light asphaltic oil has been used to a rather limited extent during the past few years as a dust palliative on gravel roads in Michigan. The oil used conforms to the following specification:

The asphaltic oil shall be homogeneous, free from water, and shall meet the following requirements:

(a) Specific gravity, 25°/25° C. (77°/77° F.)—not less than	0.900
(b) Flash point, not less than	80° C. = 176° F.
(c) Specific viscosity (Engler, 50 cc. at 25° C.)	15 to 30
(d) Loss on heating at 163° C., 20 gm., 5 hrs., not over	30 per cent
(e) Total Bitumen, not less than	99.8 per cent

Note.—This oil does not require heating for application.

Application was made by means of a pressure distributor at the rate of approximately $1/5$ gallon per square yard for the initial application early in the summer, and this was followed, after an interval of from four to six weeks, with a second application of about $\frac{1}{8}$ gallon per square yard. The ideal condition for application of this treatment is to have a thin coating of gravel, ranging in size from $\frac{1}{2}$ to $\frac{1}{8}$ inches, uniformly distributed over the surface. The use of this oil does not contemplate the formation of a mat, and in fact, every effort is made to avoid forming a mat. Subsequent maintenance is conducted with patrol graders which float the thin cover of loose material daily and do the necessary cutting of the surface to eliminate chatter-bumps. The two applications mentioned are usually sufficient for satisfactory dust laying over a period of approximately three months. The relation between the amount of oil applied and the amount of loose gravel on the road surface is important, and an excess of oil will result in a glazed surface which soon becomes very rough under traffic. The results which may be obtained with light oil treatments are comparable to those which may be obtained with calcium chloride. The cost of these treatments has varied between $300.00 and $400.00 per mile per season.

3. Bituminous Mat Surface Treatments

Rhode Island Practice. The use of a 45 per cent cutback asphaltic oil in surface treating old waterbound macadam or gravel roads is reported. The treatment consists of $\frac{1}{3}$ gallon of asphaltic oil per square yard, covered with 8 to 10 pounds of clean, sharp sand. Two to three weeks after treatment, the surface is gone over with a light planer which removes material from the high spots and deposits it in the holes where it is compacted by traffic. The average cost per mile of this treatment for 1929 is reported as $562.20. This type of treatment is applied on roads which are from ten to twenty-five years old.

A process in Rhode Island is also reported indicating that earth roads are covered with a four to six inch compacted course of gravel and then treated with $\frac{3}{4}$ gallon per square yard of cold application tar in two applications. This is followed after an interval of from six to eight weeks, with an additional seal coat of $\frac{1}{4}$ to $\frac{1}{3}$ gallon per square yard of a 65 per cent to 85 per cent asphaltic oil, and covered with clean, sharp sand, or stone chips. The cost of this process is indicated from $3000.00 to $5000.00 per mile. Such roads are reported to be carrying from 8000 to 9000 vehicles per day with vehicular weights up to 28,000 pounds.

Pennsylvania Practice. Bituminous Surface Treated Oil Bound Macadam. Oil bound macadam is constructed by placing about two inches of ¾ to ⅛-inch stone over prepared base, which may be either slag, gravel or stone. Upon the stone so spread, there is applied about ¾ of a gallon of bituminous material, in two applications. It is necessary to surface treat the roads so constructed oftener than the heavier type bituminous surface treated macadam. That is, the oil bound macadam should be treated every two years with about ¼ of a gallon of bituminous material to the square yard and about 25 pounds of chips. The cover material ranges in size from ⅝ inch to 1¼ inches.

Pennsylvania Practice. Bituminous Surface Treated Macadam. Bituminous surface treated macadam consists of a waterbound macadam sealed with a bituminous surface treatment. The initial surface treatment is made immediately following construction and thereafter treatments are required approximately every three years. At the time of construction the macadam is bound with screenings and water and is opened to traffic for a short time. The road is allowed to cure under traffic for one to three weeks until the excess screenings are worn off, then a treatment is given using 0.3 of a gallon bituminous material per square yard, which is not covered. After this material has penetrated, a second application of 0.2 of a gallon per square yard is given and is covered with approximately 25 pounds of commercial ¾-inch stone ranging in size from ⅝ to 1¼ inches. The successive treatments comprise applications each of ¼ gallons per square yard of bitumen and covering of 25 pounds per square yard of commercial ¾-inch stone.

Indiana Practice. Indiana treats, each year, several hundred miles of bituminous macadam roads, treatments being applied generally about every other year. Cover material on these treatments is dragged to increase the smoothness of the road. In treating new waterbound macadam roads, Indiana has used to some extent two applications of liquid asphalt at the rate of approximately ⅓ gallon per square yard each.

West Virginia Practice. West Virginia is continuing the work of placing asphaltic surface treatments on shale and gravel roads. This form of maintenance is regarded as successful on these types.

The U. S. Bureau of Public Roads Reports as Follows. A light oil surface treatment of crushed rock or gravel traffic-bound macadam consists of impregnating the upper portions of the consolidated metalling with a light fuel oil. The roadbed is first thoroughly cleaned with brooms to remove all loose material and expose a firm pitted surface. Light oil is applied under pressure at the rate of 0.2 of a gallon per square yard and left to penetrate. Two or three days may be required for satisfactory penetration during which time the road should be closed to traffic, and at this stage any minor imperfections in the surface may be repaired with a lean mixture of oil and suitable aggregate. A second application of oil, at the rate of 0.2 of a gallon per square yard is then applied and allowed to penetrate for several days without traffic. Stone chips in the amount of 10 to 25 pounds per square yard, depending upon the viscosity of the oil, are then applied.

Road oils, heated or cold, are used in forming thin wearing mats or carpets on macadam pavements which may be maintained or built up by subsequent treatments as desired. Successful treatments of this class require that the road surfaces shall be in good state of repair, well consolidated, free from holes or depressions, and must be thoroughly swept for the removal of dust and detritus before the applications are made. Newly constructed pavements should be permitted to thoroughly consolidate under traffic before the oil is applied. The oils may be applied either cold or hot according to the class selected, and the initial applications should be made by means of pressure distributors at the rate of from one-third to one-half gallon per square yard. It should then be covered with a uniform layer of dustless broken stone, all of which will pass the ½ or ¾-inch laboratory screen, or with pea gravel, which will pass a ½-inch laboratory screen in an amount just sufficient to absorb the bitumen and prevent the surface from picking up under traffic. Subsequent applications may be by a pressure distributor or by brooming the oil uniformly over the road surface. From 0.1 to 0.2 gallon will usually prove sufficient, and this should be covered with coarse sand or stone chips which will pass a one-inch laboratory screen.

When hot oil is used it should be heated to a temperature of 200° F. to 250° F. To secure best results the treatments should be applied in the early summer except in localities where snow and freezing weather are not found. Under favorable conditions the cold treatment should last throughout an entire season, while the hot application method should last for several years with the necessary occasional patching.

Michigan Practice. Waterbound macadam roads are surface treated by first sweeping the surface to an exposure of larger stone, and then making two applications of approximately ¼ gallon per square yard each, using, in general, a cold application surface treatment tar. Stone chips, or pea gravel, in the amount of approximately 20 pounds per square yard, are used as a cover on the second application. Gravel roads are treated in practically the same manner, being prepared in the early spring by shaping to proper cross section with a heavy grader and possibly scarifying some portions. Proper drainage and subgrade conditions are very essential in the successful surface treating of gravel roads. Tars are preferred for initial applications on either macadam or gravel roads, but asphalts have been successfully used in treatments other than first year treatments. Hot application materials are also considered preferable, excepting for initial treatments, as it is believed that there is less inconvenience to traffic where hot application materials are used.

California and Oregon Practice. Bituminous surface treatments are applied to tightly bound crushed gravel and rock, or screened gravel roads. Asphaltic oil heated to a temperature of 150 to 200° F. is applied with a pressure distributor. The road surface is prepared by sweeping with a revolving broom until the dust is removed and the imbedded rock exposed. The first application is made at the rate of $\frac{1}{4}$ gallon per square yard, and is allowed to penetrate for a period of from 6 to 24 hours. The second application of $\frac{1}{4}$ gallon per square yard is then made, and this is covered with an application of stone chips or pea gravel, from $\frac{3}{4}$ to $\frac{1}{8}$ inch in size. Proper drainage and subgrade conditions are considered very important in this work. Subsequent maintenance consists of skin patching in the shallow holes, or the use of cold-patch mixtures.

4. Bituminous Retread and Other Mixed-in-Place Types

Indiana Practice. Bituminous retread top, as constructed in Indiana, consists of placing a course of stone, two to four inches in thickness, on a prepared gravel, or macadam, base. This stone may range in size up to $2\frac{1}{2}$ inches. It is first shaped with a grader and rolled to proper cross section. The first application of bituminous material is applied and allowed to cure, after which the surface is rolled and any inequalities corrected by use of a grader. The second application of bituminous material is then applied and allowed to cure, after which the surface is again rolled. An application of stone chips is then placed on the surface and thoroughly broomed into the voids. A third application of bituminous material is then placed, a covering of stone chips added, and the surface thoroughly dragged with a long base drag or planer. The surface is then given a final rolling and is opened to traffic. An additional seal coat of bituminous material may be necessary at a little later date if the surface appears to be too open.

Michigan Practice. Retread tops, two inches in thickness, are constructed in Michigan on gravel or macadam bases, according to the following method; the gravel or macadam base, if it has not been previously surface treated, is given a prime coat of bitumen at the rate of about ¼ gallon per square yard. Stone, ranging in size from ⅝ to 1¼ inches, is placed on the prepared base to such depth as will compact to approximately two inches. This aggregate is then brought to proper cross section with graders and is given an application of about ¾ of a gallon per square yard of bitumen. The stone is then bladed back and forth across the road during the time the bituminous material is setting up. This blading process is continued until the stone is thoroughly coated with bitumen, at which time the mixture is bladed to proper cross section and rolled. A seal coat of about ¼ gallon per square yard is then applied, followed by an application of stone chips and additional rolling. The cost of this method is approximately 50 cents per square yard for two-inch thickness.

West Virginia Practice. A two-mile section of retread on a gravel base has been placed in West Virginia and this surface has given satisfactory service for three years under a rather heavy truck traffic.

The U. S. Bureau of Public Roads reports the following general procedure in the construction of retread without use of a roller:

1. Scarify and reshape old base, adding additional stone if necessary, or merely sweep base clean of dust and dirt, if it is of proper cross section and grade and well solidified.
2. Spread layer of stone about 2 inches thick (use ¾ to 1½-inch stone).
3. Treat with 0.5 to 0.75 gallons per square yard of bitumen.
4. Blade surface to proper cross section.
5. Continue blading to maintain desired cross section until the surface begins to set.
6. At the end of four or five days after the surface has set, apply about ¼ gallon of bitumen followed by light cover of stone chips. Traffic is permitted to operate continuously during the entire construction.

The U. S. Bureau of Public Roads reports the following practice in a mid-western state on the construction of retread, using a roller:

1. Scarify and reshape old base if necessary or merely sweep it clean of dust and dirt.

2. Spread layer of stone to the desired thickness (2, 2½ or 3 inch loose, use 1¼-2 inch stone) (1 inch loose, use ¾ to 1¼ inch stone).
3. Roll stone lightly.
4. Apply first treatment of bitumen (about 0.4 gallon per square yard).
5. Blade immediately, only disturbing top layer.
6. Roll immediately after blading and again after two days.
7. Fill surface voids with stone chips applied sparingly.
8. Apply light treatment of bitumen after first treatment has set (about 0.2 gallon per square yard).
9. Drag surface, preferably with light drag.
10. Roll chips into surface after dragging.
11. In about two weeks apply still lighter coat of bitumen (about 0.15 gallon per square yard).
12. Cover sparingly with chips and roll thoroughly.

The thickness of layer and size and quality of the stone used, as well as the quantities of bitumen applied, will necessarily vary under different traffic and other pertinent conditions.

There are numerous procedures followed in the processing of mixed-in-place roadbeds; one method consists essentially of scarifying, shaping and pulverizing the old surface after which the bituminous material is applied in one or more applications by means of gravity or pressure distributors. The surface is disced slightly after each application. This, as a rule, proves valuable in preventing loss of the light constituents of the bituminous material. Mixing is carried on by means of the blade grader, the procedure being to carry the treated material from one side of the road to the other. This process is continued until a uniform mixture is secured, after which the material is evenly spread, shaped and allowed to compact under traffic.

While processing the existing surface of roadways is at times successful, it should be avoided provided sufficient funds are available to build up a new course for treatment. Working direct on the existing surface necessitates breaking up the wearing surface or natural crust, uncovering at times unstable areas which fail after the mixed course is formed. For this reason it is best to leave the crust formed on the traveled way undisturbed where possible, and spread for mixing purposes a layer of the best material at hand.

California Practice. Description is given of a mixed-in-place treatment which is usually applied to a base which is metalled either with crushed rock or gravel, the binder of which is deficient in cementing qualities. The base is scarified to a uniform depth of three to four inches, and then harrowed. A fuel oil heated to 150 to 200° F. is then applied with a pressure distributor at the rate of approximately 1½ gallons per square yard. The road is then harrowed, after which oil is mixed with the aggregate by blading the mixture back and forth across the road. This mixing is continued until the aggregate is quite thoroughly coated, after which it is bladed to proper cross section and opened to traffic. Dragging is continued over a period of about seven days to eliminate any ruts which may be produced by traffic. Obstructions are placed in the middle of the roadway to force traffic to use the edges in order to thoroughly compact them.

5. Penetration Macadam and Similar Types

West Virginia Practice. A modified type of bituminous macadam called "Puddle Macadam" is constructed in West Virginia. The method consists of placing a four-inch course of stone on a prepared base and rolling same just enough to shape and compact the loose stone, but not enough to crush or break the stone. Stone screenings and dust are then spread over the surface to a depth of about ¼ of an inch over the top of the stone, and this is then rolled and dust added where necessary until no movement is discernible under the roller. The surface is then swept clean of any surplus screenings or dust, and an application of cold tar is made at the rate of one gallon per square yard. A cover of stone chips is then added and the surface is again rolled. Traffic uses the road at all times except for a few minutes when the bituminous material is being poured and the cover coat applied.

Pennsylvania Practice. Bituminous Penetration Macadam. In the construction of bituminous penetration macadam, the aggregate used for the lower course is stone ranging in size from 1¼ to 2¾-inch, and an application of 1.85 gallons per square yard of asphaltic oil is given. Then a covering of 60 pounds per square yard of commercial ¾-inch stone, which is followed by a treatment of 0.3 of a gallon per square yard of asphaltic oil. After this treatment has penetrated sufficiently, an application of 0.35 gallon of bituminous material is given and covered with 30 pounds per square yard of ⅛ to ⅝-inch chips.

In the final application of 0.35 gallons per square yard, tar is used instead of asphalt in order to improve tractive quality of surface. Penetration macadams require successive treatments at four or five-year intervals after construction. These treatments of bituminous penetration macadam are the same as successive treatments of bituminous surface treated macadam and oil bound macadam.

Michigan Practice. Bituminous macadam by the penetration method. A 3-inch course of stone ranging in size from ½ to 3 inches is placed on a prepared macadam base, and spread uniformly to proper cross section. The stone is then thoroughly rolled and irregularities corrected during the process of rolling. Hot bituminous material is then applied with a pressure distributor at the rate of 0.7 to 0.8 of a gallon per square yard, per inch of thickness. Coarse chips are then applied and thoroughly broomed into the voids. The surface is then thoroughly rolled and a seal coat of bitumen applied at the rate of ¼ to ½ gallon per square yard. This is followed by a cover of fine chips and additional rolling. A second seal coat of about ¼ gallon per square yard is applied, preferably a few months after the road has been placed under traffic. Fine chips are used to cover the second seal coat.

California Practice. A type of surface course called "Armor Coat" is used in California. This treatment consists of two separate applications of heavy asphaltic road oil on an adequately metalled and traffic-bound base, following a prime coat of light fuel oil. The roadway is first swept with a power broom until the dust has been removed and the imbedded stone exposed after which the prime coat is applied and allowed to thoroughly penetrate the base. The first application of asphaltic road oil is then made at the rate of 0.2 to 0.25 gallons per square yard and is followed with an application of 30 to 40 pounds of stone chips per square yard. The surface is then rolled with a six-ton roller. The surface is again swept and a second application of asphaltic oil is placed at the rate of 0.2 of a gallon per square yard. Thirty to thirty-five pounds of stone chips per square yard are then applied and the surface rolled. The result is a non-skid surface, which is successfully carrying traffic up to 2000 vehicles per day. The cost of the treatment is reported as approximately 20 cents per square yard.

Oregon Practice. A so-called "Multiple Lift" type of surface is used to some extent in Oregon. It consists of light courses of 1½ to ¾-inch crushed rock, oiled and rolled, which, after the desired thickness has been secured is oiled and sealed with stone chips. The oil is first applied and after covering with stone, is levelled with a blade grader and then rolled. Each lift is complete in itself, with the exception of the seal coat, and the use of the blade grader insures a very smooth job.

Indiana Practice. Indiana reports constructing in 1929 more than 100 miles of rock asphalt surface on penetration macadam. Standard penetration macadam is built on any suitable base, using only 1¼ gallons of bituminous material per square yard for 4¾-inch depth of loose aggregate. This leaves a very lean, open surface, on which is placed from 80 to 100 pounds of rock asphalt per square yard. By a suitable levelling and planing process, the surface is made very smooth and uniform. The service record of this type of surface does not extend over a sufficient period of time to draw definite conclusions on maintenance upkeep, but there is every indication that, under normal traffic, the maintenance cost will be very reasonable. Tractors with lugs, and heavy horse-drawn traffic, damage this type during the warm season, and it should not be constructed where such traffic is heavy.

6. Hot Mixed Asphaltic Surface Courses

Michigan Practice. During the past six years Michigan has laid approximately 113 miles of 18-foot width asphaltic surfaces on gravel and macadam bases. This work consists of stone filled sheet asphalt laid to a depth of 2 inches, and of sheet asphalt laid to a depth of 1½ inches. The original gravel or macadam base is widened where necessary with gravel, crushed stone, or slag, and given a prime coat of bitumen applied at the rate of about ¼ gallon per square yard. A black base mixture is then placed and rolled for the purpose of eliminating high crown, or any irregularities in the base, as well as to strengthen the base. The top course of sheet asphalt, or asphaltic concrete, is then placed and rolled. The average cost of top course for five years' work is $1.04 per square yard. The average cost per mile of this construction, including shoulder work, is $17,300.00. Work is performed with state-owned asphalt plant and force account organization.

FORMATION OF WASHBOARDS IN GRAVEL HIGHWAYS

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INTRODUCTION

Highway transportation in the United States is becoming more important and more popular each year. In many places where the traffic will warrant the expenditure, hard surface roads have been built. Whether or not a cement concrete or bituminous type road is built in a given locality is determined partly by the traffic need for such a road, and partly by the financial conditions and the political situation of that region as well. In places where hard surface roads for one reason or another cannot be built, and traffic warrants an improved road, it has been customary to surface the road with gravel or crushed rock with which a small amount of clay is mixed for binder. This is a type of surface commonly found in the Pacific Northwest. There are thousands of miles of such highway in use throughout the United States with many additional miles being added each year.

A crushed rock or gravel type surface when in good condition gives excellent service for both passenger and trucking traffic. However, as the volume of traffic reaches one thousand or more cars per day, such a surface quickly develops into what is commonly called "washboards." This phenomenon is also known as rhythmic corrugations, ripples, chatter bumps, etc.

The presence of "washboards" in gravel or crushed rock highways has proven to be a serious problem in many ways. For one thing, it increases the hazard of driving in that cars cannot be so well controlled over such highways at high speeds. The severe treatment endured by the tires and running gear of the car tend to shorten the life of those parts materially. In the case of trucks, a badly washboarded highway tends to limit the maximum possible load which the springs can carry. On account of these problems, the highway maintenance engineers have continuously sought means whereby they could either eliminate or materially diminish the formation of "washboards" in gravel highways. Their efforts have been directed primarily toward the treatment of the road surface, and this has been only partly successful. By the use of intensive maintenance such as frequent planing and blading of the gravel surface, they have been able to limit the depth to which " washboards " will form. However, the fact that the road surface is being continuously stirred and pulverized by the maintenance equipment has augmented what has popularly been called the " dust menace " of driving. For these reasons and in the interests of economy it is desirable if possible to learn if there may not be some other way of combating the " washboard " menace than by means of an intensive maintenance program.

With this situation in mind, the Engineering Experiment Station of the State College of Washington a number of years ago undertook a study of the cause and prevention of " washboards " in gravel highways. In 1926, a car was equipped with a recording mechanism which recorded on paper the relative motion of a car axle with respect to the body of the same car when traveling over " washboarded " roads. Extensive tests were made at different speeds over several different " washboarded " highways. Tests were also conducted on perfectly smooth roads on which has been placed a bump or a single depression. This made it possible to analyze the entire history of a single road shock. It was soon apparent that in the making of these tests there were other factors besides the bumps on the road surface which were entering into the test record. One of these especially was the lurch of the car body from side to side. During the summer of 1927, a small laboratory apparatus was developed which embodied the characteristics of a car and on which tests could be made without the introduction of disturbing influences other than those being studied. All cars possess the characteristics of sprung and unsprung weight. The car body, which is carried on the springs is the " sprung weight." The car axle together with the hubs, brakes, and wheels constitute the " unsprung weight." This " unsprung weight " however, is somewhat cushioned by the air in the tires. By means of the above mentioned laboratory apparatus, careful analysis was made of the relative motion of the different parts of a car when subjected to " washboard " shocks at different speeds. The results of these tests were published in Engineering Bulletin No. 19.

In order to verify, the conclusions in the above report, during the following summer of 1928 a test track was built on the State College campus for carrying on further " washboard " tests. This track was built 175 feet in diameter and banked for a normal speed of 20 miles per hour. The surface of the track was given a gravel topping according to standard highway specifications except that it was only about five inches thick. A model T Ford touring car, used for the test, was equipped with 3½-inch high pressure tires, at fifty-five pounds air pressure and driven at 20 miles per hour. " Washboards " could be formed on the gravel surface of the test track in 100 trips. When equipped with Houdaille hydraulic shock absorbers, the formation of " washboards " was found to be much slower. In fact, it was not possible to make satisfactory " washboards " on the test track with this combination. When balloon tires were substituted without shock absorbers, it was found impossible to make " washboards " on the test track even though the test was continued until the road was virtually worn out.

While the above tests all contributed to the fund of information on the subject of " washboards," it was felt that further tests should be conducted on a straight highway. This would more nearly duplicate the conditions which exist on the public highways, and results from such tests would be more readily accepted by engineers and others interested in highway maintenance.

OBJECTS OF TESTS

The result of the foregoing tests and experiments would seem to indicate that there is a possibility of solving the " washboard " problem in so far as it can be solved not by attacking it from a maintenance standpoint but by controlling the conditions and characteristics inherent in the car itself. If this were possible, then at least part of the present objectionable and expensive method of continuously loosening the road surface would be eliminated. A reduction of this maintenance program would save substantial amounts of maintenance money, which could be otherwise devoted to improving the highways. With these things in mind, the problem was laid before the State Highway Department of Washington. Later, with their cooperation, a stretch of gravel road was chosen and prepared for further tests during the summer of 1929.

IDEAL CONDITIONS FOR TESTS

During the long dry summers which are characteristic of eastern and central Washington, road surfaces become more or less thoroughly dried out. The dry clay binder which is a part of the gravel and crushed rock highways found in this section loses its cementing value under the force of heavy automobile traffic. The continued impact of automobile tires tends to loosen and move this gravel surface into successive bumps and hollows. If this gravel is restored to a smooth surface by means of a drag or grader, an equal amount of traffic will again restore the "washboard" condition. For this reason, the dry summer weather of Eastern Washington makes it possible to duplicate conditions and repeat tests with fair uniformity of results.

DESCRIPTION OF TEST ROAD

Out of several sites under consideration, one was chosen which is located about twelve miles southwest of Spokane. This road was a part of the original highway between Spokane and Cheney before the new concrete road was built. It runs east and west on the east side of the concrete highway and ¼ mile north of Four Lakes, Washington. The road is level throughout its length except for a short section which has a grade of from three to five per cent. There are two long easy curves. The surface appears to be of screened material from a glacial gravel deposit, and it is estimated that 85 per cent of this loose material on the road would pass a ⅜ inch screen. The sand is quartz with practically no basalt present. During the season of the spring rains, the State Highway department went over this road with a grader and left it perfectly smooth. Under the normal light traffic (10 to 15 cars per day), the surface became hard and smooth. During the following dry weather, a grader was again used to plane the surface leaving about two inches of loose material on a perfectly smooth hard base.

CONDUCTING TESTS

Preliminary to each test, this loose material was brought to the center of the road and spread evenly a little wider than the width of a car. The drag used (see Figures 1 and 2) consisted of four angle-irons rigidly welded to cross-bars and dragged behind the car at about sixty degrees angle to the line of travel. After going over the road with the drag in one direction, the angle of the drag was reversed on the return trip. This tended to eliminate "drag ripples" and left the road surface practically smooth and free of any bumps or depressions. See Figures 3 and 4.

After a test had been completed, "washboards" would be present at various places on the road. See Figure 5. Owing to the light traffic, the test car was driven forth and back in the same track, thus

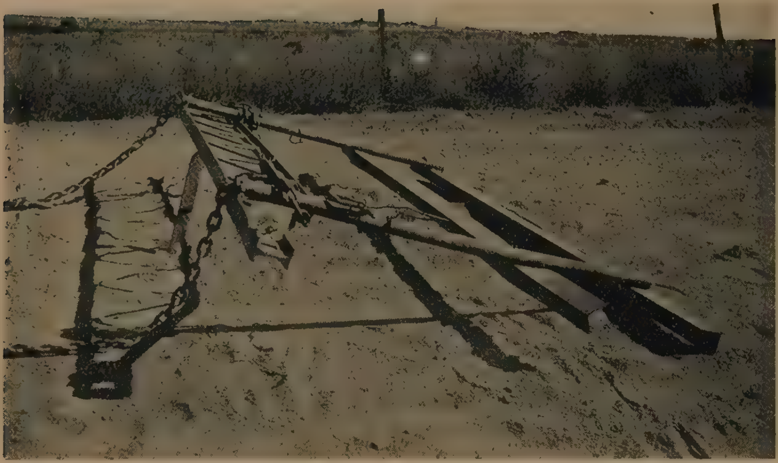


Figure 1. The Road Drag Used to Prepare the Test Road for the "Washboard" Tests, the Drag is Shown Lifted from the Road Surface to Reveal the Four "Edges"

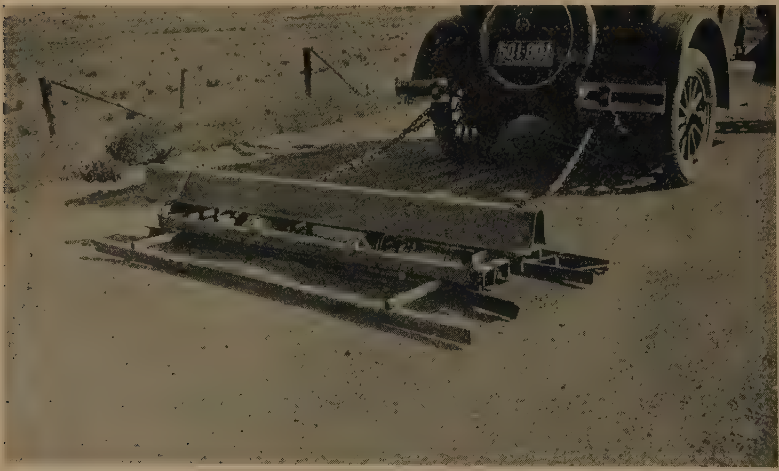


Figure 2. The Road Drag Fully Weighted and Attached Behind Test Car, as Shown, the Blades of the Drag Make an Angle of from 50° to 60° to the Line of Travel

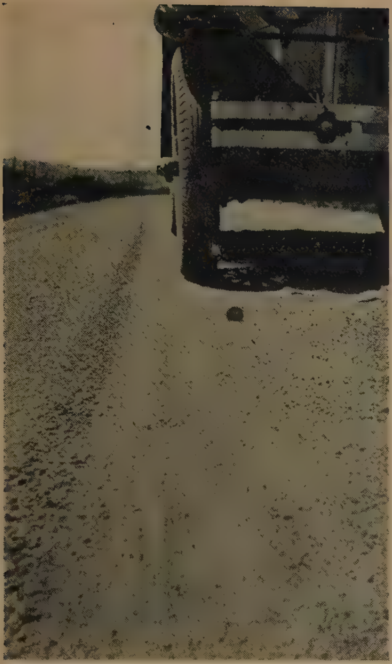


Figure 3. View of Test Road on the 25 M. P. H. Section During Test with Balloon Tires

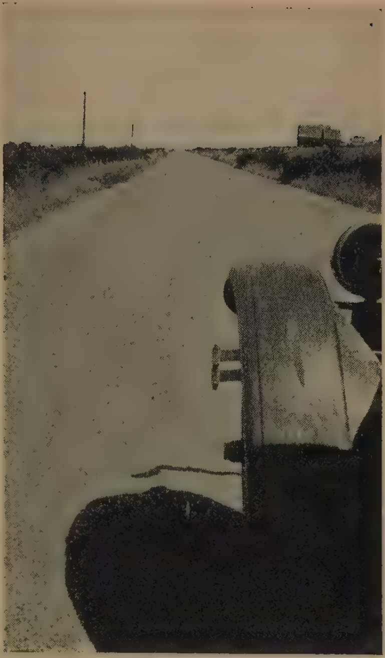


Figure 4. Note the Smooth Surface of Gravel Packed by the Balloon Tires. No Evidence of Washboards Here.

Figure 5. "Washboards" as Developed on the 35 M. P. H. Section, Using High Pressure Tires

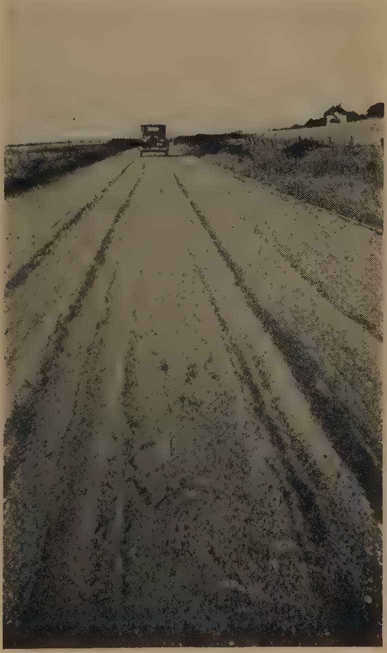


Figure 6. Showing the Progress of the Test in the Loose Gravel on the 40 M. P. H. Section, Using Balloon Tires

Figure 7. View on the 40 M. P. H. Section Showing the Gravel Smoothed and Compacted by Balloon Tires

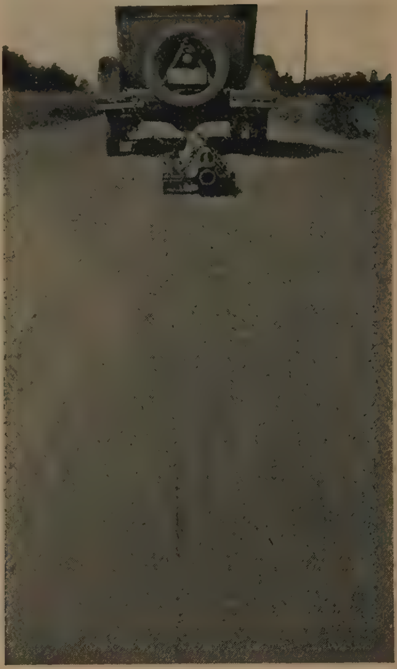


Figure 8. The Weighted Scarifier in Action. Note That It Has Thoroughly Loosened the Compacted Gravel

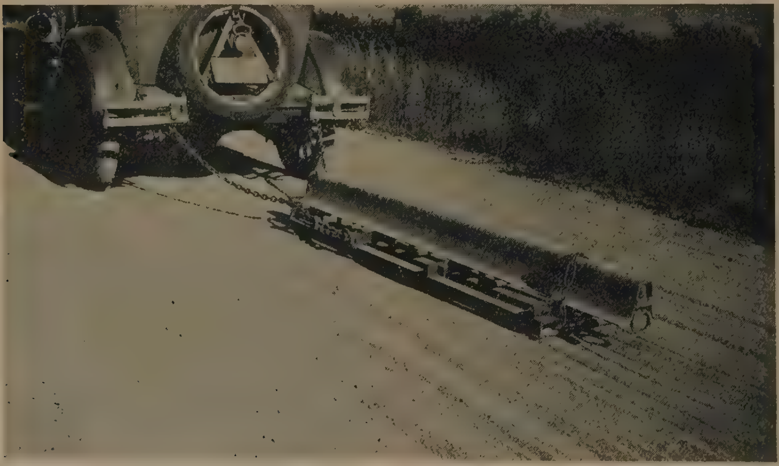


Figure 9. Quarter View of the Weighted Scarifier in Action to Erase the Effects of the Previous Test on the Road Surface

concentrating all the travel on the center of the road. See Figures 6 and 7. In order to thoroughly erase the influences of any previous test, the road was scarified before it was dragged. The scarifier consisted of a steel frame about eighteen inches wide and four feet long. See Figures 8, 9, 10. Four rows of teeth were arranged equi-distant along the length of this frame. Successive rows of teeth were staggered with the preceding rows. While this scarifier was not heavy enough to actually tear up the hard surface of the road, it thoroughly loosened the gravel which the car had packed into "washboard" shape during the previous test. Thus after scarifying and dragging

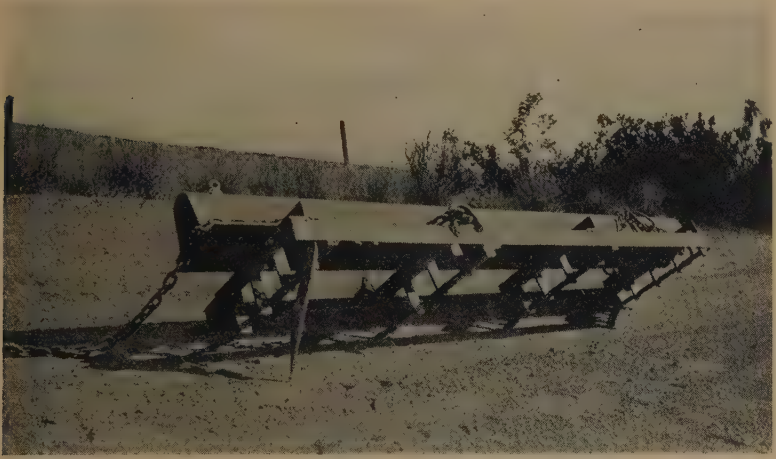


Figure 10. Scarifier Raised from the Road Surface to Show the Four Rows of Teeth. Each Row Is Staggered with the Preceding Row. View Shows Teeth Nearly Worn Out

the surfaces for a new test, it was reasonably certain that the results would not be influenced by what had taken place previously.

DESCRIPTION OF TEST CAR

It was deemed desirable that the car for these tests should be of medium size and weight. The car chosen was a 1927 Chevrolet Coach of 103 inch wheel base and weighing 2620 pounds with driver. Figures 11 and 12. Two sets of wheels were provided. On one set were mounted balloon tires, size 29 by 4.40 inch and operated at 34 pounds air pressure. On the other set of wheels were mounted high pressure tires, size 30 by $3\frac{1}{2}$ inch and operated at 55 pounds air pressure. The car was equipped with a new set of Two-Way hydraulic shock absorbers. Figure 13.

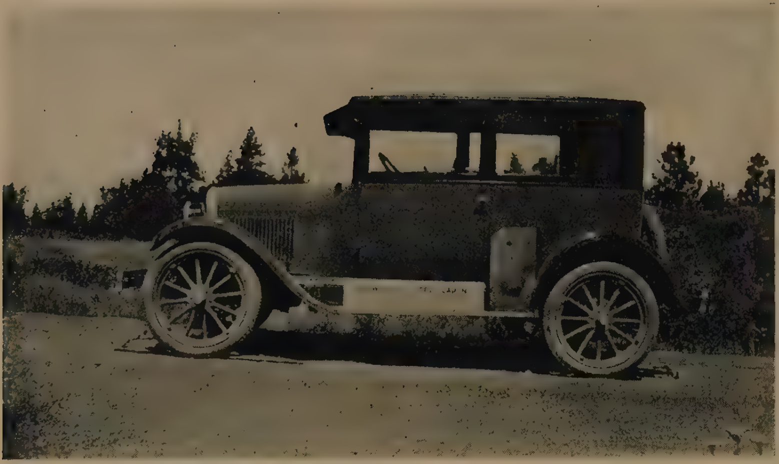


Figure 11. View of Highway Washboard Test Car of the Engineering Experiment Station, Car Is Shown with High Pressure Tires.

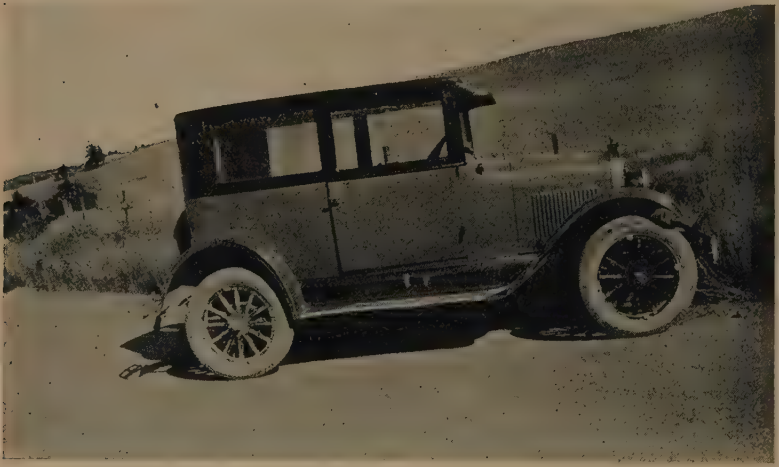


Figure 12. View of Test Car Equipped with Balloon Tires. Separate Wheels Were Provided for the Two Types of Tires

DESCRIPTION OF TESTS

The test road which was 1.4 miles long was divided into five sections. On the section at one end of the road a speed of 25 miles per hour was maintained in either direction. On the adjacent section, the speed was raised to, and maintained at, 35 miles per hour. On the next section, at 40 miles per hour; on the next section, at 30 miles per hour; and on the other end section, at 25 miles per hour. At each end of the track, a short section was reserved for decelerating prior to turning around and for accelerating again to 25 miles per hour. Also between adjacent sections, it was necessary to reserve a short distance for accelerating or decelerating for the speed in the next section. Thus when the test was completed comparative results were shown at four different speeds.

At the beginning of the test, several days were spent in learning how to develop and control the desired conditions. For instance, it was found that dragging must be done at a speed not exceeding three to four miles per hour. Otherwise, the drag would jump and chatter leaving a rough surface. It was also found that unless the angle of the drag to the line of travel was reversed each time over the road, a series of drag ripples would result which would seriously influence the results of the tests. Repeated tests were made during these first days until it was felt that the conditions were understood and could be controlled and duplicated at will. This made it possible to put the track in the same shape for each successive test. Furthermore, it was discovered that the formation of " washboards " as far as comparative tests were concerned need not progress to the extreme conditions found on average gravel highways. In other words, a stopping point for the tests was determined upon in which the " washboards " were perhaps not more than $\frac{1}{8}$ of an inch deep, and had not dug into the hard road surface. They consisted entirely of the loose gravel placed and packed in more or less uniformly spaced shallow ridges and hollows. At this stage of their formation, the " washboards " were plainly visible to a driver in a car, and the distance between successive ridges could readily be marked and measured.

The driver of the test car was constantly on the alert during the progress of the test to observe the first appearance and location of " washboards." Note was made of the time of occurrence and the section of the road on which " washboards " first appeared. As might be predicted, they first appeared on the highest speed section, and were last to appear on the lowest speed section. Furthermore, it was observed that the average length of "washboards" varied according to the speed of travel on that section.

RESULTS OF TESTS

Table I shows the summarized results of the tests on high and low pressure tires with and without shock absorbers.

TABLE I

Formation of Washboards as Related to Speed of Travel on Test Track, 1929

			Trips to make "washboards"			
Test No.	Type tire	Shock absorbers	25 M. P. H.	30 M. P. H.	35 M. P. H.	40 M. P. H.
1	High Pressure	No	100	91	36	36
2	High Pressure	No	No results			
3	High Pressure	No	137	79	71	65
6	High Pressure	No	146	76	60	50
5	High Pressure	Yes	No W. B. at 290	230	153	230
7	High Pressure	Yes	188	188	105	92
9	High Pressure	Yes	No W. B. at 350	220	126	110
4	Low Pressure	No	No W. B. up to 360 trips			
8	Low Pressure	Yes	No W. B. up to 570 trips			

It will be noted that Two-Way hydraulic shock absorbers were used on the 1929 tests. These shock absorbers, Figure 13, were installed and used with the original factory adjustment which gives slight resistance to compression and considerable resistance to recoil. While tests could have been made with other adjustments of the shock absorbers it was felt that other means could be devised for determining the optimum adjustment without spending the days and weeks necessary to conduct the road tests. In Table II are shown the average lengths of the washboards as measured on the different speed sections of the test track. In Figures 14, 15, 16, and 17 are shown some typical washboards found on other highways during the summer of 1929. They average about 1 inch in depth. In Figure 18 are shown the curves giving the information contained in Table I.

CONCLUSIONS

From Table I it will be seen that speed of travel bears a definite relation to the rapidity with which "washboards" are formed by high pressure tires. Furthermore, the addition of one type of shock absorber at least has a retarding effect on the formation of "washboards" and would seem to prevent their formation at low speeds. It

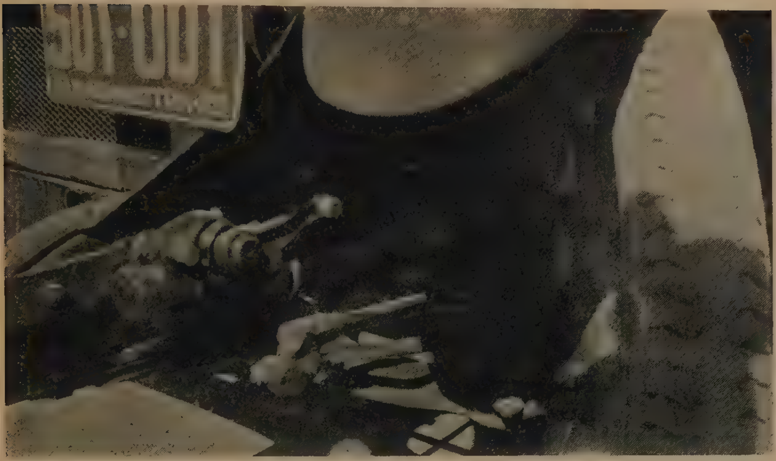


Figure 13. View Showing Installation of Two-Way Hydraulic Shock Absorbers on the Test Car. The Car Was Equipped with a Set of Four Shock Absorbers

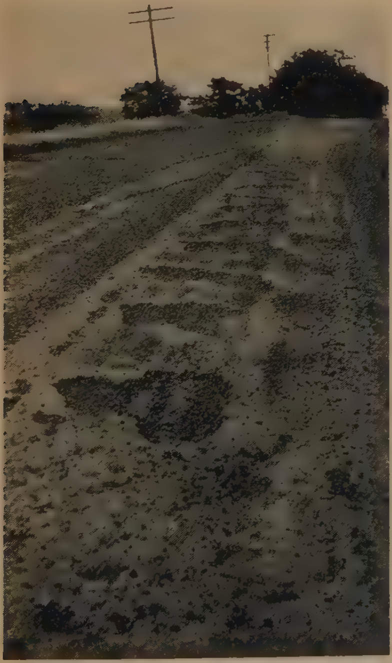


Figure 14. View of "Washboarded" Section of Gravel Highway Which Had Been Treated with Light Oil to Lay the Dust. Surface of Road Was Fairly Smooth and Hard. View Taken about Three Weeks after Application of Oil

Figure 15. "Washboards" Formed in Dry Loose Gravel. Base Was Hard and Nearly Smooth, with Humps Formed of Fine and Coarse Material More or Less Compacted Into Shape but Still Loose and Easily Moved

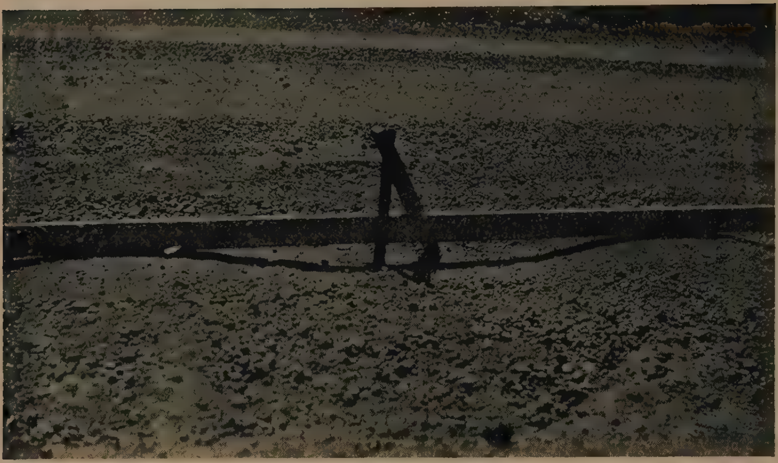


Figure 16. View of "Washboards" in Oiled Gravel Highway, Bumps Are Hard, and about 1 Inch Deep. These "Washboards" Were Formed by Digging Pits Into the Hard Surface of the Road. Not Much Loose Gravel Is Present

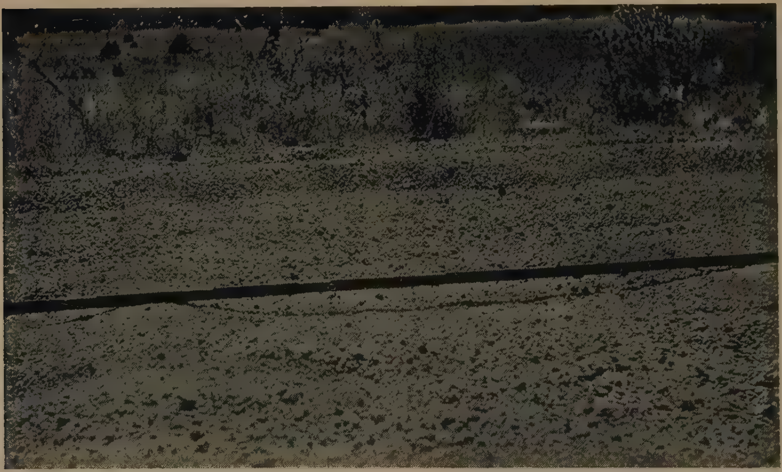


Figure 17. "Washboards" in Dry Loose Gravel. The Bumps Are of Loose Material Which Can Be Scuffed Away with the Foot. The Hollows Are about 1 Inch Deep

will be noted that in these tests, balloon tires did not make " washboards " even after prolonged operation on the test track. Whether or not, balloon tires would promote or retard their formation on an already, deeply " washboarded " highway is a question yet to be answered. Likewise, the comparative effectiveness of different types of snubbers and shock absorbers is still to be determined.

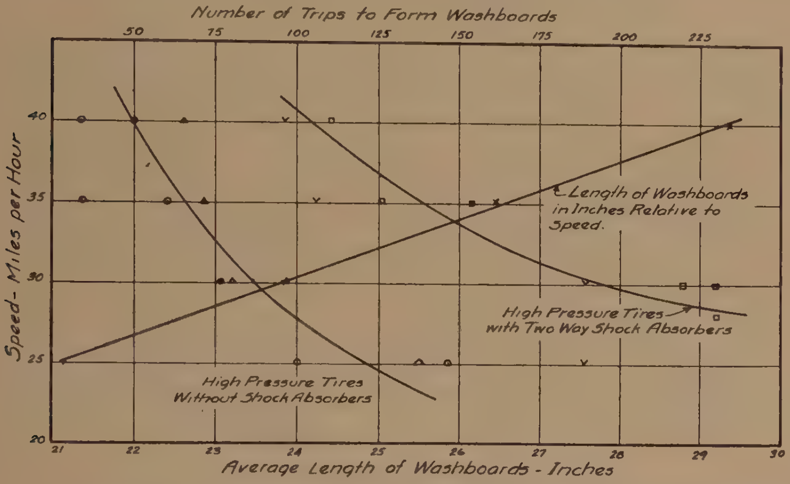


Figure 18. Formation of Washboards as Related to Speed. Chevrolet Coach with 30 x 3.5 Inch Tires at 55 Pounds Pressure and 29 x 4.4 Inch Tires at 34 Pounds Pressure. No Washboards Could Be Made with Low Pressure Tires Either with or without Shock Absorbers in 360 and 570 Trips

From Table II it will be noted that the average length of " washboards " in the different speed sections varies somewhat according to the speed of travel of the car used in the tests. If the movement of the wheels and axles had corresponded with the free movement of a weighted spring, then the third column would have shown one constant value as the rate of vibration regardless of speed of travel of the car. The actual vibrations as computed from the average length of the " washboards " and the speed at which they were formed gives

TABLE II

CHARACTERISTICS OF WASHBOARDS MADE ON TEST TRACK, 1929

Speed M. P. H.	Average length of W. B. in inches	Number W. B. passed per sec.
25	21.14	20.8
30	23.83	22.1
35	26.47	23.3
40	29.32	24.0

values ranging from 20.8 to 24.0 vibrations per second. This indicates that the speed of travel of the car introduces certain factors which cause the wheels and axles to vibrate at a rate other than their "natural period."

While the percentage of high pressure tires on passenger cars has greatly diminished and is at present very small, the percentage of trucks on the highways is constantly increasing. Truck tires are, at present, essentially of the high pressure type, although a very few trucks are using what is known as the truck type balloon. It is hoped that subsequent tests in the laboratory and on the highway will yield definite information as to how much the behavior of the car or truck wheel can be controlled with reference to its propensity for making "washboards." We are encouraged to believe that some type of snubber or shock absorber applied to the high pressure type of tires will not only contribute to the easier riding quality of the vehicle thus equipped but will serve to greatly diminish the expense of surface maintenance. Such an accomplishment would point the way to a possible saving in maintenance money in the State of Washington alone of thousands of dollars per year.

STUDY OF TEST CAR

In order to more correctly interpret the results of the tests described, an intensive study was made of the behavior of the test car itself.

In the car was mounted a rotating cylinder, geared to the speedometer drive, and carrying a blank paper record. Two pencils were arranged to record on the rotating cylinder, the relative motion between the front and rear axles and the car body. Motion was conveyed to these pencils by means of steel piano wire run over pulleys and attached as near as possible to the right hand wheels. The wire was attached to the pivot pin on the right front axle and to the brake housing on the right rear axle, both points of attachment being about the same distance from the plane of the tire.

With the above equipment, the test car was driven over a relatively smooth and hard gravel highway, at one point of which had been placed a rounded artificial bump 6 inches wide and 1½ inches high. This was placed in the path of the right hand wheels only. Records were made of the relative motion of the axles with respect to the car body at 20, 25, 30, 35 and 40 miles per hour, using high pressure tires, and low pressure tires, both with and without two-way hydraulic shock absorbers. These records are shown in Figures 19, 20, 21 and 22.

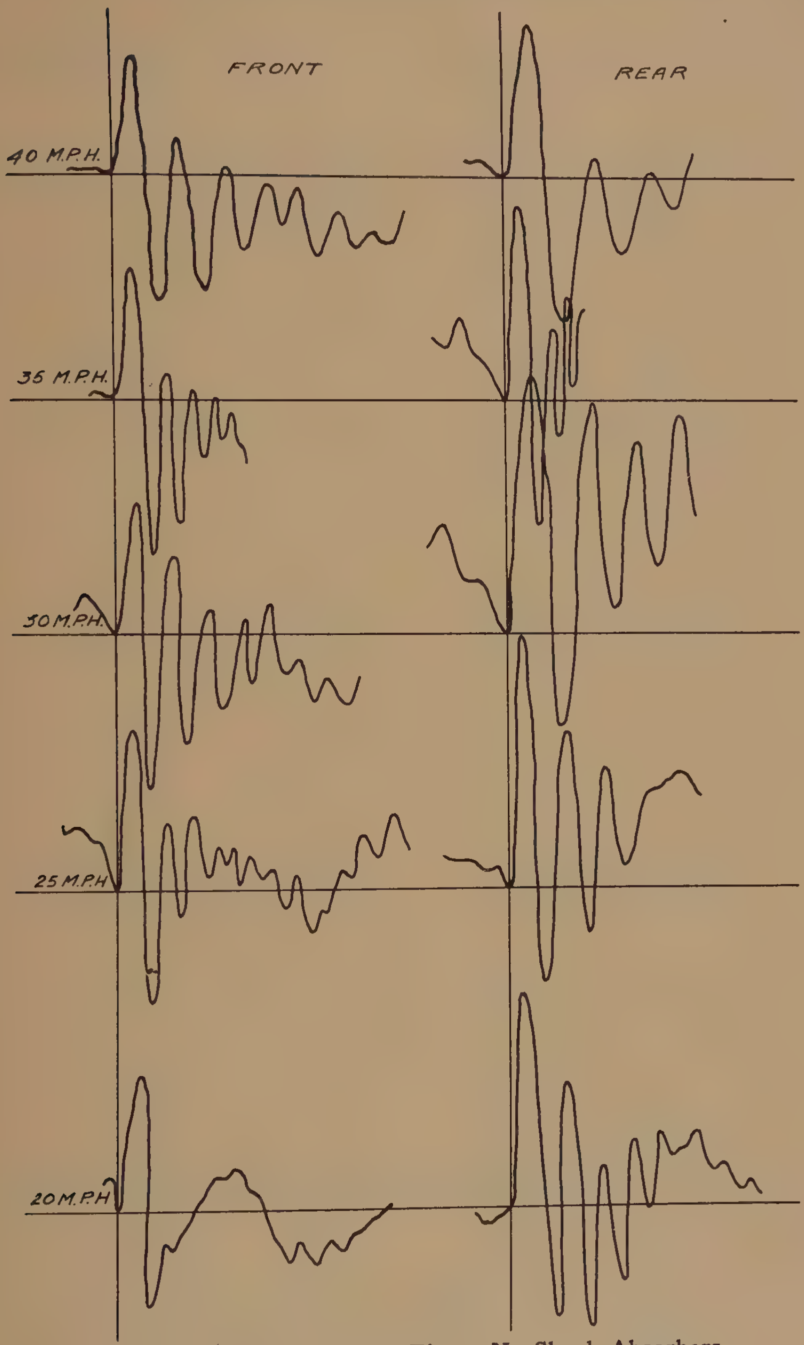


Figure 19. High Pressure Tires—No Shock Absorbers

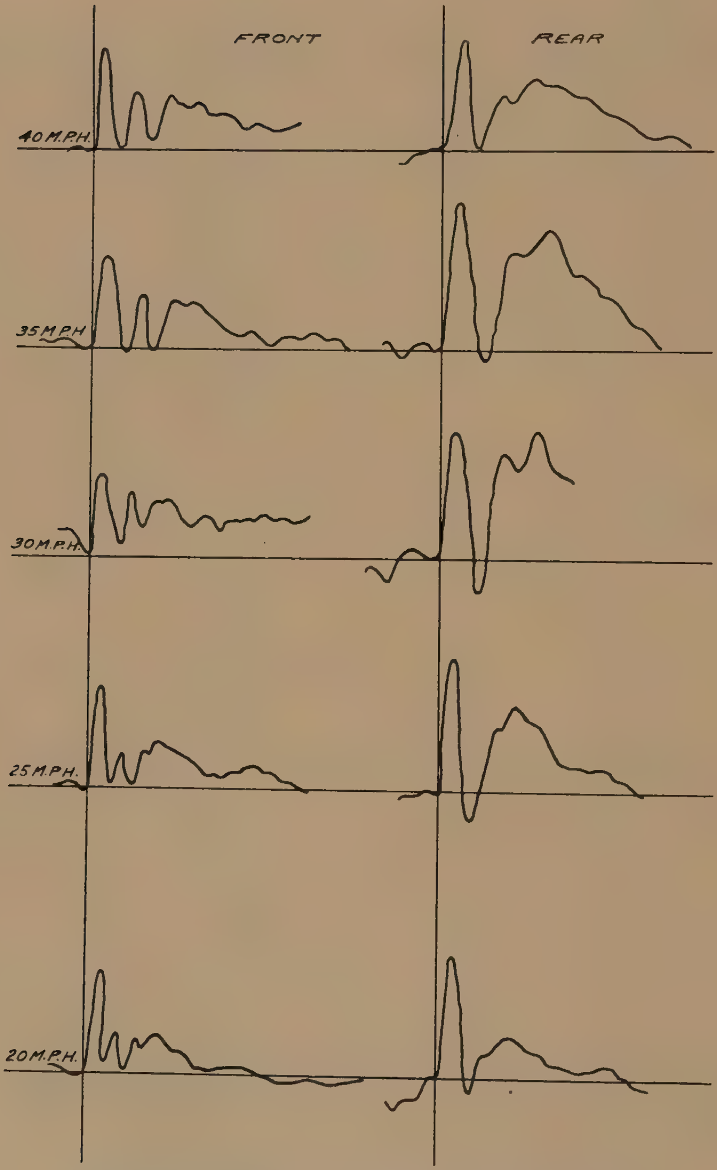


Figure 20. High Pressure Tires—Shock Absorbers

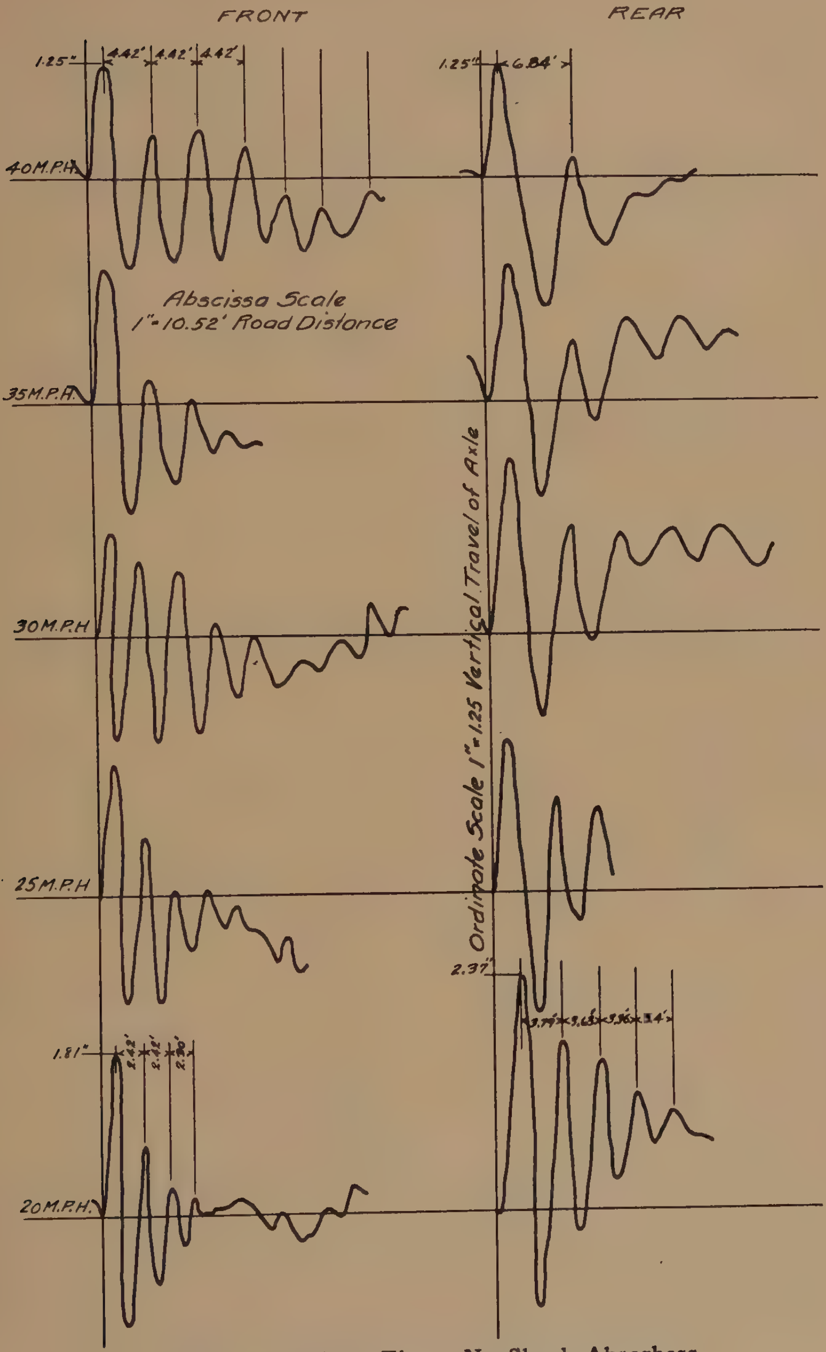


Figure 21. Balloon Tires—No Shock Absorbers

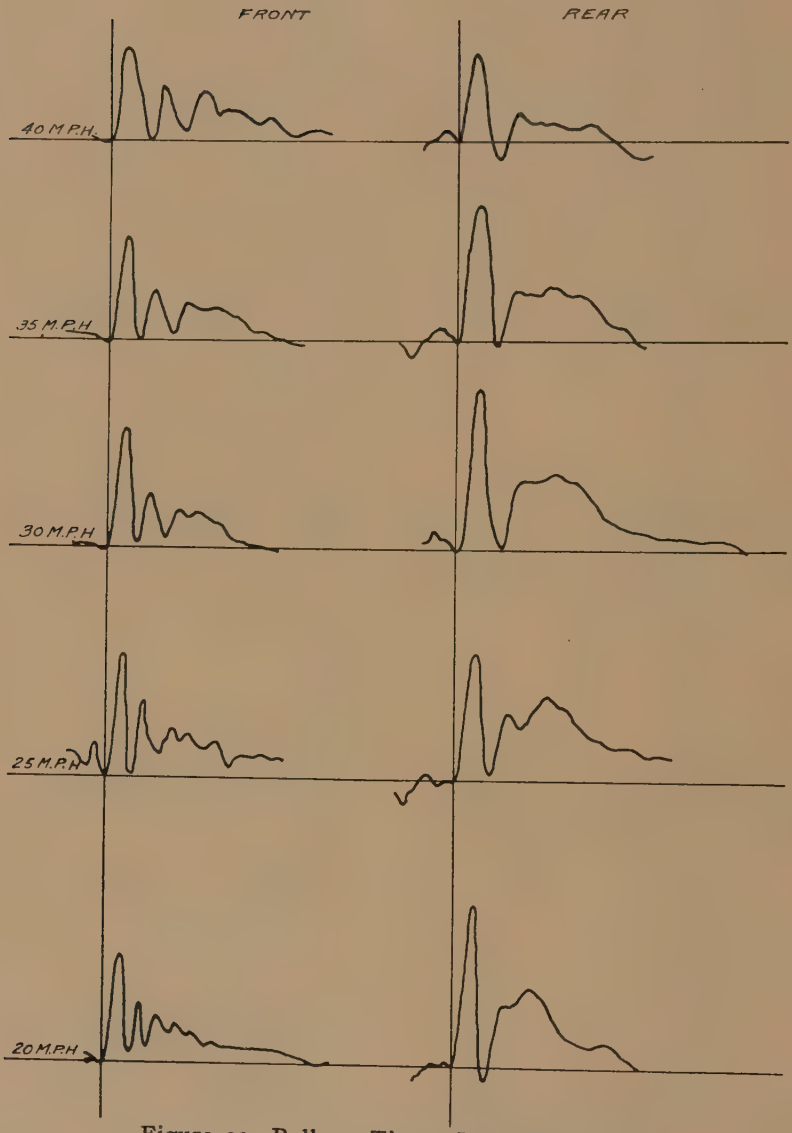


Figure 22. Balloon Tires—Shock Absorbers

ANALYSIS OF RECORDS

A general inspection of these records reveals that the use of shock absorbers either with high pressure tires or with balloon tires not only quickly damps out the axle vibrations but also reduces their amplitude of vibration. This means that the tire itself probably takes more of the shock due to the fact that the retarded recoil of the springs by the snubber action of the shock absorbers causes the car body to ride "lower" with reference to its axles on "washboarded" roads. This undoubtedly is one of the things which influence the increasing of the rate of vibration of the axles with an increase of speed of the car, as indicated in Table II.

The records also show that the rate of vibration is higher for the front axle than for the rear axle. For instance, in Figure 21, at 40 M. P. H. the rate of vibration beginning with the first recoil is calculated to be 13.25 per second for the front axle and 3.57 per second for the rear axle. The amplitude with respect to the car body of the first vibration of the axle upward after striking the obstruction is 1.25 inches front and 1.25 inches rear at 40 M. P. H. This amplitude increases with decrease of speed showing that the tire probably absorbs more of the road shock at high speeds.

Also, from Figure 21, at 20 M. P. H. the rate of vibration is 12.2 per second for the front axle and approximately 7.9 per second for the rear. The amplitude with respect to the car body of the first vibration of the axle upward after striking the obstruction is 1.81 inches front and 2.57 inches rear at 20 M. P. H.

From the above it will be noted that the apparent distance between impacts of the tire on the road surface after striking the single obstruction exceeds the average length shown for corresponding speeds in Table II.

In this connection, consider in Figure 23, the action of the axles of the test car traveling at 20 M. P. H. over a "washboarded" highway where the average length of the "washboards" is found by measurement to be 23.9 inches. Neither the front axle nor the rear axle are oscillating in step with the "washboards" being traveled over, although both are not far from synchronism. Where the car is traveling at 30 M. P. H. (Figure 23) the front axle is shown to be oscillating practically in synchronism with the "washboards" and the curve is remarkably regular. On the other hand, the rear axle is oscillating more or less independently of the "washboards" and at a slower rate.

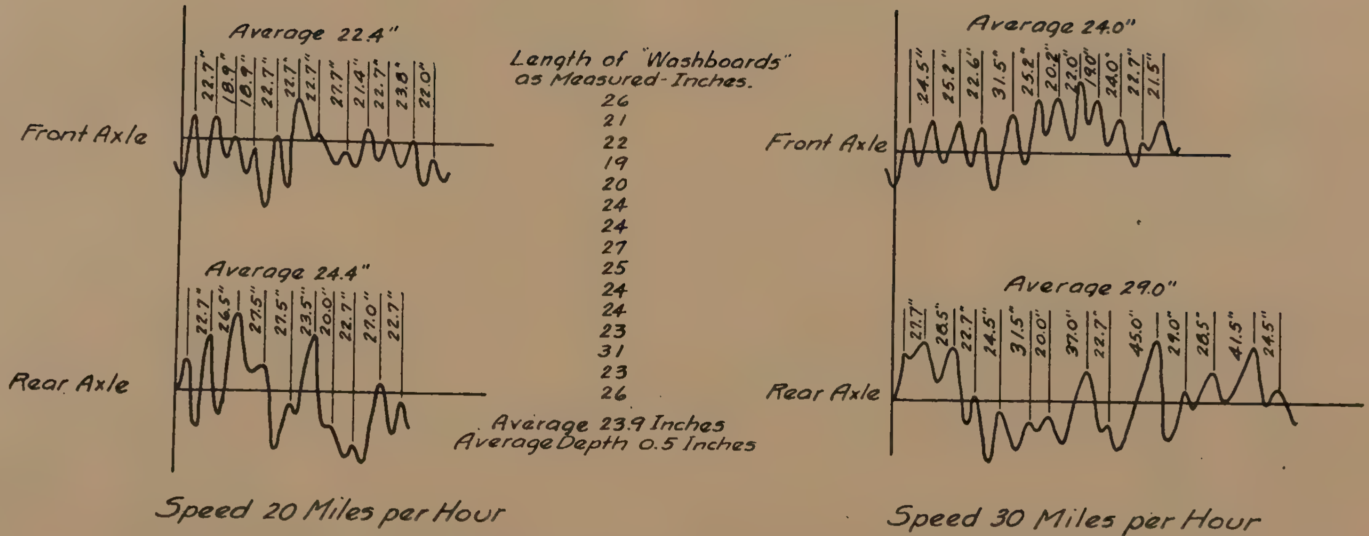


Figure 23. Test Car Driven Over a "Washboar ded" Road at Two Speeds. High Pressure Tires with Shock Absorbers. Not on Test Track

When the axle vibration does correspond with a "washboard," then the reaction is indicated by a peak in the curve higher than the average.

From the information secured thus far, it would appear that the average length of the "washboards," formed on each section of the test track as shown in Table II, must represent a compromise between, or a combination of, the separate forces exerted on the road surface by the two vibrating axles of the car, and therefore does not correspond to the natural period of either. This indicates that further study is necessary in order to fully understand the relation between cause and effect.

OUTLINE OF FURTHER STUDIES

The data thus far secured seem to indicate that there are a number of questions relative to the prevention of "washboards" which are yet to be answered. For instance, will balloon tires driven over a badly "washboarded" highway tend to build up or tear down the existing "washboards?" Will the addition of shock absorbers to a balloon tire equipped car change the answer to the above question?

The results of previous tests indicate that high pressure tires are far more distructive of gravel road surfaces then are balloon tires. Since high pressure tires on passenger cars are fast disappearing, our attention is necessarily directed toward high pressure tires on trucks. This type of traffic is rapidly increasing both in weight and numbers. Much of the stage traffic would be classed with trucks as regards the high pressure tires. The question is, what is the relative effect of this type of traffic upon the highways considering the load per square inch tire contact with the road surface and the customary speed of travel?

In the light of the tests just concluded, at least one particular type of shock absorber (when applied to a passenger car) serves to delay the formation of "washboards" on gravel highways.

The question is, what type of shock absorber is most efficient in the prevention of "washboards"? Furthermore, to what extent can the formation of "washboards" be controlled by the application of such shock absorbers to trucks and stages equipped with high pressure tires? And again, is it possible that a more efficient type of shock absorber can be developed which will not only give riding comfort, but will be more effective in the prevention of "washboards"?

These and other kindred questions have presented themselves in the course of this investigation and the Engineering Experiment Station proposes a continued study of this problem which will ultimately embrace all of the questions mentioned above. This will involve an intensive laboratory study of the inherent characteristics of different cars, to be followed by extensive tests on certain standard gravel highway surfaces. As rapidly as time and finances will permit it is proposed to carry this problem to completion.

REPORT OF COMMITTEE ON STRUCTURAL DESIGN OF ROADS

A. T. Goldbeck, *Chairman*

Director, Bureau of Engineering, National Crushed Stone Association

A glance back over the past decade reveals great activity in the field of research in the structural design of roads. Problem after problem has appeared, each of great importance and the solution of which, when extended over the thousands of miles of highways constructed, has resulted in a very substantial saving.

At the beginning of the building of the Federal Aid Highway System some thirteen years ago, many doubts existed in the minds of highway engineers as to what constituted proper methods of road design. There were few scientific facts established applicable to the design of roads, constructed of necessity under an extremely wide range of climatic conditions; supported on a foundation, now a swamp and now solid rock; carrying traffic having a wide range of characteristics, changing rapidly in character from year to year and ever increasing in intensity.

The problems requiring solution were so numerous and so vital that the initiation of research on any one of them was apt to be productive of important results. Moreover, there were comparatively few highway research agencies and consequently no particular need existed for a far reaching and comprehensive plan of investigation laid out for the purpose of distributing the work. There were more important highway questions crying for solution than could possibly have been undertaken by all of the highway research organizations in the country.

Perhaps those investigations which have been performed in the structural design field have seemed haphazard, but actually this has not been the case. The research agencies have always had their eyes on those goals whose attainment meant most in the possibility of effecting telling results.

To those who may inquire as to just what the research accomplishments have been and to question as to whether the energy expended has been worth while, let them remember the decided improvements in concrete road design, the thickened edge, the center joint and the establishment of the functions and the proper use of reinforcing steel. Let them compare the advancing knowledge of subgrades and drainage and the far-reaching effects resulting from the practical application of this information. Let them reflect on the extreme value of the researches in low cost roads and the establishment of the limitations of their usefulness. Let them compare the design of present day motor truck tires with those of ten years ago. Is it not evident everywhere how the trend is toward pneumatic tires as against the old, thin, solid tires with their heavy impact effects? Is it not becoming increasingly apparent how numerous are the vehicles having multiple wheels which decrease the load intensity and better distribute it over the road surface? Unquestionably, these advancements in heavy vehicle design are producing a saving of millions of dollars in our costs of highway upkeep. Intensive research is the foundation upon which these changes have been based.

A huge system of highways is unique in lending itself to the benefits of research, for if an advanced method of design or construction is thereby established, the resulting gain is felt not merely in a short stretch of road; it is multiplied time and again, over thousands and thousands of miles of roads to be constructed or to be maintained in the future. It is this fact which makes highway research of such vital importance. Lack of knowledge, resulting in a comparatively small error in design, when repeated over miles of highways will mean an economic loss of millions of dollars and a corresponding saving, not only is possible, but actually has been made as the result of that work which thus far has been accomplished.

Is it not clear, then, that highway research must go on and now that the agencies and their facilities have multiplied is it not equally clear that systematization, which will focus research efforts on those problems having the highest importance, must produce even more fruitful results than have been obtained in the past?

Your Committee on the Structural Design of Roads firmly believes that research in its field should proceed. Judging from the outline which we are now studying and perfecting the potential advancement in the design of highways seems without limit.

In the past much attention has been directed with admirable effect to the concrete type of road, partly because of the wide-spread use of this popular and deserving type and partly, too, because concrete is a more or less elastic material, which engineers understood and could theorize upon much more readily than with materials of a plastic nature. Research in other types has lagged but changing economic conditions and the seeming necessity of building roads of lesser importance but of huge mileage makes it desirable that more effort be devoted to the improvement of suitable, but less expensive designs. But the more costly types still have as high importance as ever and this fact is not overlooked in your committee's program of work.

It would be a dreary and boresome performance were I to go into detail regarding our research program which now is being completed. I shall content myself with a bare outline of its contents. There are five main divisions including:

I. Subgrades and Drainage.
II. Traffic.
III. Climate.
IV. Road Types.
V. Special Problems.

Under each of these there appear a multitude of research projects, some of which have already been worked upon and partially solved, but a majority of which have not been completed or even investigated. Upward of a thousand topics have been listed and no doubt some of them may be subdivided into many more.

So the field of research in the structural design of roads is extensive and there is abundant opportunity for the Highway Research Board to bring about a coordinated and cooperative system of highway research which should net results of tremendous value and importance. There is need for just such an agency as the Highway Research Board for marshalling the increasing investigative efforts of the country and a well-laid out program now seems quite essential to the accomplishment of that end.

THE INFLUENCE OF CLIMATE ON THE BUILDING, MAINTENANCE AND USE OF ROADS IN THE UNITED STATES

F. H. Eno

Research Professor of Highway Engineering, Ohio State University

It is the province of Sub-Committee No. III of the Committee on Structural Design of Roads, of the Highway Research Board, to prepare an outline of the information regarding the various phases of climate that affect the building, the maintenance, and the use of highways.

Much of this informatioin is available through the published works of the U. S. Weather Bureau. Many do not know, however, in what an acceptable form it has been prepared and presented by that Bureau.

This committee will attempt therefore, from time to time, to present the salient features of the Weather Bureau reports that affect in any manner the building and use of highways, and to outline further lines of research that will be necessary to complete our knowledge of the effects of climate upon roads.

Your committee has tentatively outlined the work to be considered and will present it under the following general heads, viz:

1. Temperature.
2. Frost.
3. Sunshine
4. Wind.
5. Humidity
6. Precipitation.
7. Run-off.
8. Evaporation.
9. Application of the data to highway activities.
10. Further lines for research.

U. S. WEATHER BUREAU PUBLICATIONS

Of the many publications issued by the U. S. Weather Bureau, the committee will confine its present report to two sets; namely, the three volumes of Bulletin W, entitled " Summaries of Climatological Data by Sections," and " Part II—Climate " of " The Atlas of American Agriculture."

From Part II, " Section A—Precipitation and Humidity " and " Section B—Temperature, Sunshine and Wind," with a few loose pages upon Frost will be briefly described.

Bulletin W summarizes the climatological data into tables. The United States is divided into 106 sections. The sections are small, many of the states being divided into three or four sections.

Within these sections the climatological data for each town of any consequence, which has a weather bureau observation station, are given in tables.

The summaries contain full tables of monthly and annual precipitation, tables of snowfall, averages and extremes of temperature, relative humidity, sunshine, wind, dates of first and last killing frosts for each year, the elevation of the observation station, and other special data.

The charts in the Atlas picture these tabulated figures in diagrammatic and map form.

As your committee has had no opportunity to search widely for climatological data it will present a preliminary report illustrating what may be found in the U. S. Weather Bureau publications. These publications may be found in all of the state libraries and most of the university and large city libraries.

TEMPERATURE

Temperature, either as heat or cold, affects practically every phase of road building, road maintenance or road use. Heat causes blow-ups of concrete and monolithic brick roads. It softens tar and asphalt treated surfaces. It seriously affects the integrity of green concrete that has not been perfectly protected. Extreme heat materially decreases the amount of work performed by labor and seriously affects its character.

Extreme cold is as adverse in its affects upon road building, or more so, than extreme heat.

Therefore, any definite information, easily available to the engineer and contractor, about any climatic factor enables them to more surely build good roads.

There are three distinct or major types of physical climate—marine, continental, and mountain. There are minor types such as coastal climate, desert climate, and the like.

Marine climate is marked by more even, uniform temperatures throughout the year.

Continental climate shows greater temperature extremes and more rapid changes.

Mountain climate has lower temperatures than the adjacent lowland areas, and greater variation between the daily maximum and minimum temperatures.

Figure 1 illustrates the annual march of temperature from January to December for selected stations in four distinctly different regions of the United States, viz:—The Atlantic Coast, the Pacific Coast, the Mississippi Valley and the Rocky Mountain area.

The engineer is deeply interested in the risks which low temperatures bring to his work. Figure 2 presents a map of the United States upon which are located lines showing the lowest observed temperatures throughout the country, based upon the records of about 600 stations. These lowest observed temperatures range from — 65° F. in eastern Montana to + 41° F. at Key West, Florida.

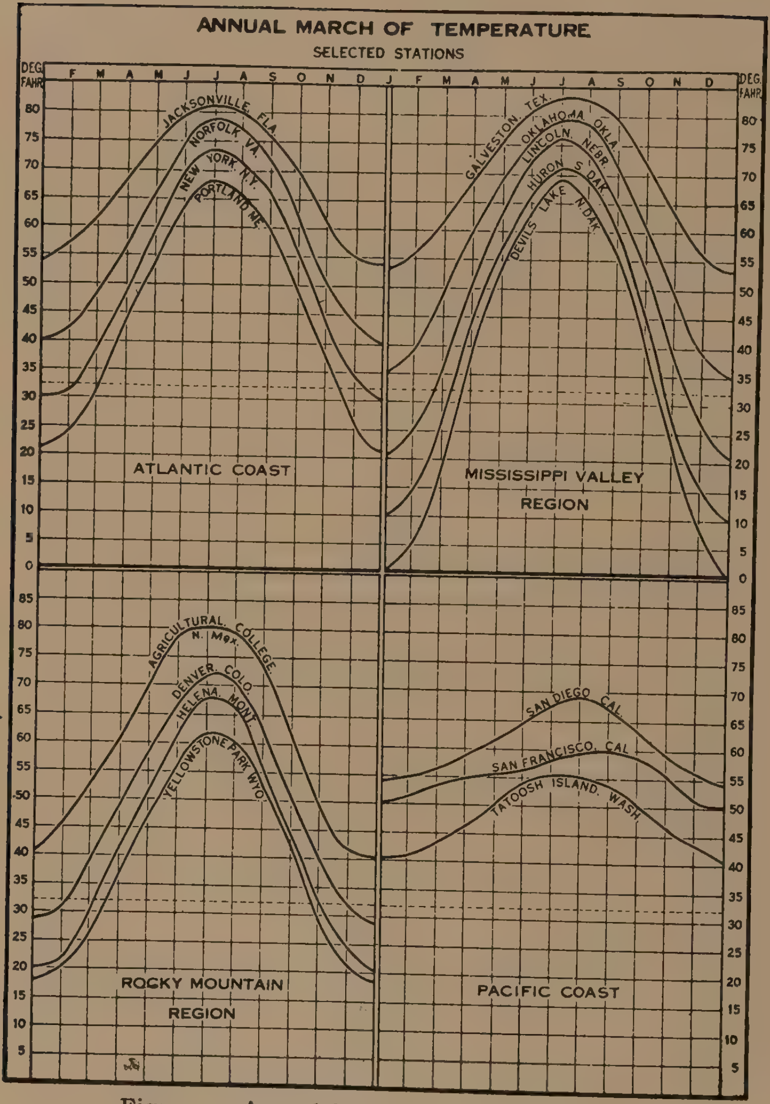


Figure 1. Annual March of Temperature

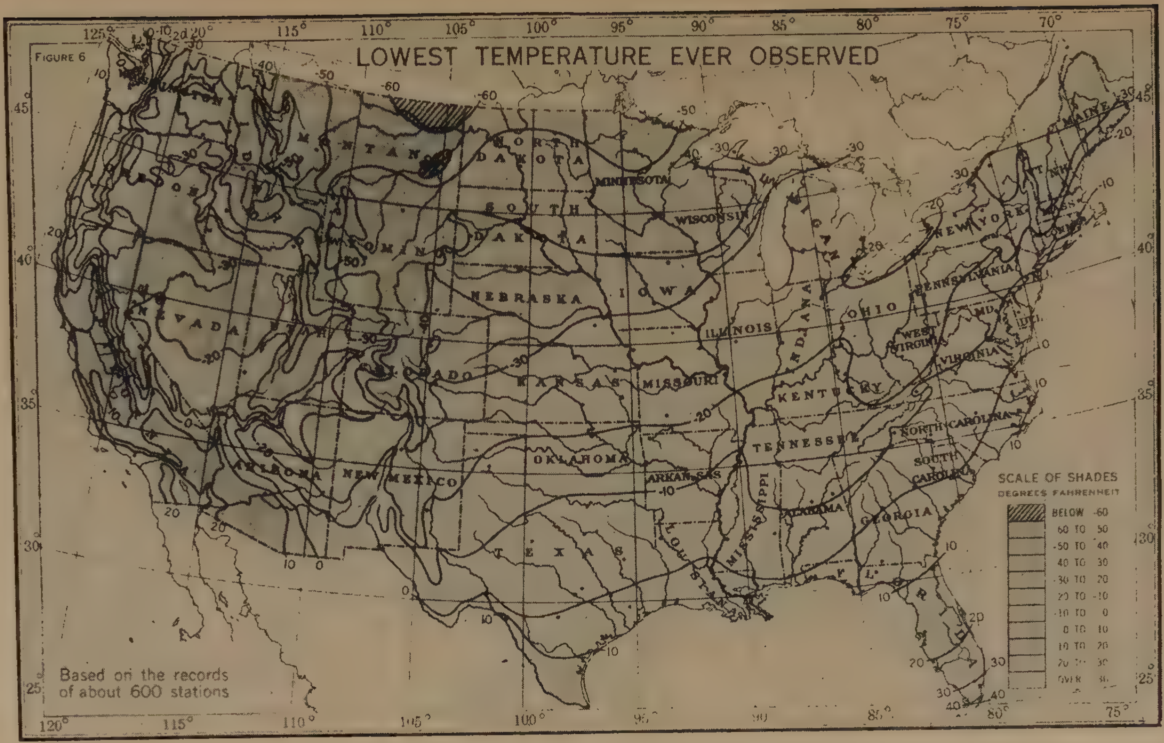


FIGURE 6
LOWEST TEMPERATURE EVER OBSERVED
SCALE OF SHADES
DEGREES FAHRENHEIT
BELOW -60
-60 TO -50
-50 TO -40
-40 TO -30
-30 TO -20
-20 TO -10
-10 TO 0
0 TO 10
10 TO 20
20 TO 30
OVER 30
Based on the records
of about 600 stations
WASHINGTON
OREGON
NEVADA
MONTANA
WYOMING
UTAH
COLORADO
ARIZONA
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MISSISSIPPI
MICHIGAN
INDIANA
KENTUCKY
TENNESSEE
ALABAMA
GEORGIA
FLORIDA
OHIO
WEST VIRGINIA
VIRGINIA
NORTH CAROLINA
SOUTH CAROLINA
PENNSYLVANIA
NEW YORK
MAINE
125°
120°
115°
110°
105°
100°
95°
90°
85°
80°
75°
70°
45°
40°
35°
30°
25°
-60
-50
-40
-30
-20
-10
0
10
20
30
40

Figure 2. This Map Shows the Lowest Temperatures Ever Observed up to and Including the Year 1922, Based on the Records of the Regular Reporting and of Selected Cooperative Stations. These Absolute Minimum Temperatures Range from —65° F. in Eastern Montana to 41° at Key West, Fla. Temperatures of —40° have been Recorded in Northern New England and Northern New York, and as Low as —20° as Far South as Tennessee, Arkansas, and Oklahoma, and Zero Temperatures Have Occurred in the Central Gulf Coast Districts. Along the Central and Southern California Coast the Lowest Temperatures of Record are from 24° to 28°

For the concrete and highway engineers and contractors, northeastern Montana is a very undesirable anti-paradise.

The Weater Bureau Atlases show mean high, low and average temperature maps of the United States for each month of the year.

Figure 3 shows the lowest monthly mean isotherms for the month of April, while Figure 4 shows the same lines for November.

By selecting a number of stations scattered over the United States and spotting them upon a U. S. Map, a valuable graph of the annual march of temperature across the country from south to north is presented.

The average daily maximum and minimum temperatures, the average monthly, the absolute minimum and maximum temperatures are all thus shown on one sheet, on Figure 72, in Section B of Part II, Climate, in the Atlas of American Agriculture.

FROST

Frost affects the work of the engineer in two ways, one in the actual depth to which freezing occurs in the ground and the other in the chill of the air that affects the rate of setting of concrete and may actually seriously damage its strength.

In the first case, the freezing of the ground delays and even totally interrupts the grading and drainage work of construction. It also causes heaving of the finished pavement from slight amounts to such an extent as to totally destroy the pavement.

In the second case, beginning at a temperature of about 45° to 40° F., temperature first retards setting concrete and later as the temperature falls it begins to damage the green concrete by freezing crystals of water throughout the mass, segregating the mortar and causing material weakness.

The committee upon its first visit to the Weather Bureau found that little had been done in the way of collecting material as to the depth of freezing in soil over the United States area.

At a second visit, however, the committee found that the Bureau had been very actively collecting such data and would have information available probably early in the summer of 1930.

Such data may be collected from Telephone and Telegraph Companies, from contractors who construct sewers, water supply mains and other subsurface construction, from Public Utility Companies and cities.

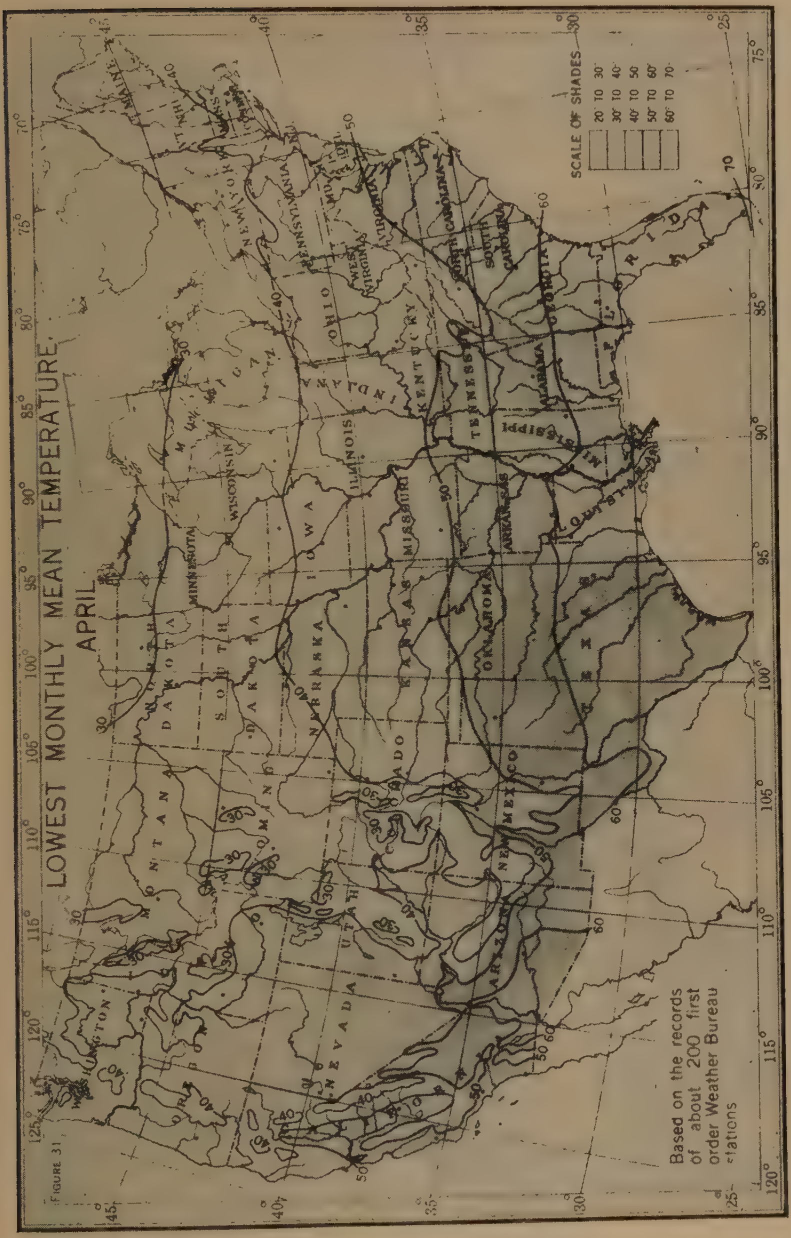
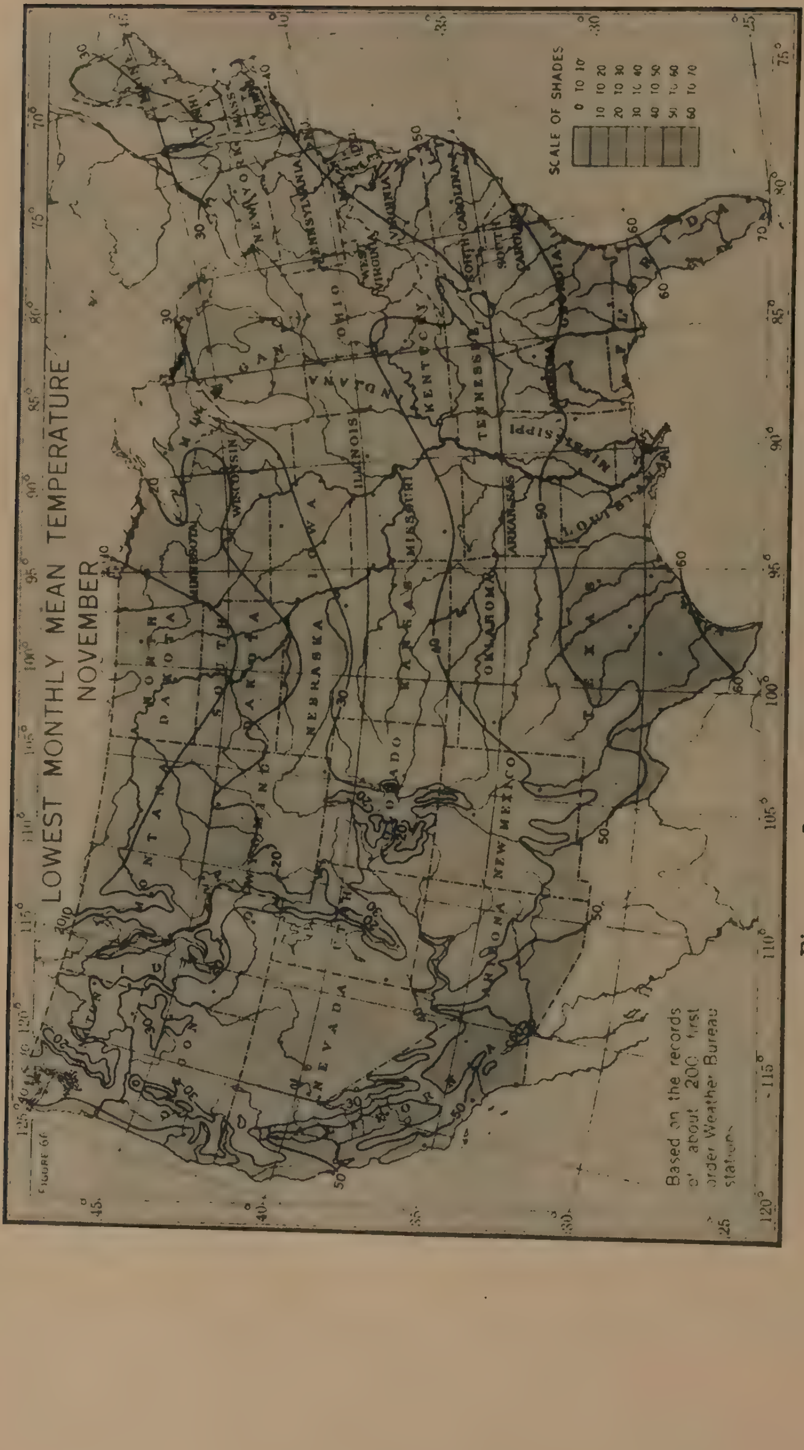

LOWEST MONTHLY MEAN TEMPERATURE, APRIL
SCALE OF SHADES
20° TO 30°
30° TO 40°
40° TO 50°
50° TO 60°
60° TO 70°
Based on the records of about 200 first order Weather Bureau stations
FIGURE 31

Figure 3. Lowest Monthly Mean Temperature, April


LOWEST MONTHLY MEAN TEMPERATURE
NOVEMBER
SCALE OF SHADES
0 TO 10
10 TO 20
20 TO 30
30 TO 40
40 TO 50
50 TO 60
60 TO 70
FIGURE 66
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
N.J.
DEL.
MASS.
CONN.
MONTANA
WYOMING
COLORADO
NEW MEXICO
ARIZONA
UTAH
NEVADA
IDAHO
OREGON
WASHINGTON
CALIFORNIA
10
20
30
40
50
60
70
125°
120°
115°
110°
105°
100°
95°
90°
85°
80°
75°
70°
45
40
35
30
25

Based on the records of about 200 first order Weather Bureau stations

Figure 4. Lowest Monthly Mean Temperature, November

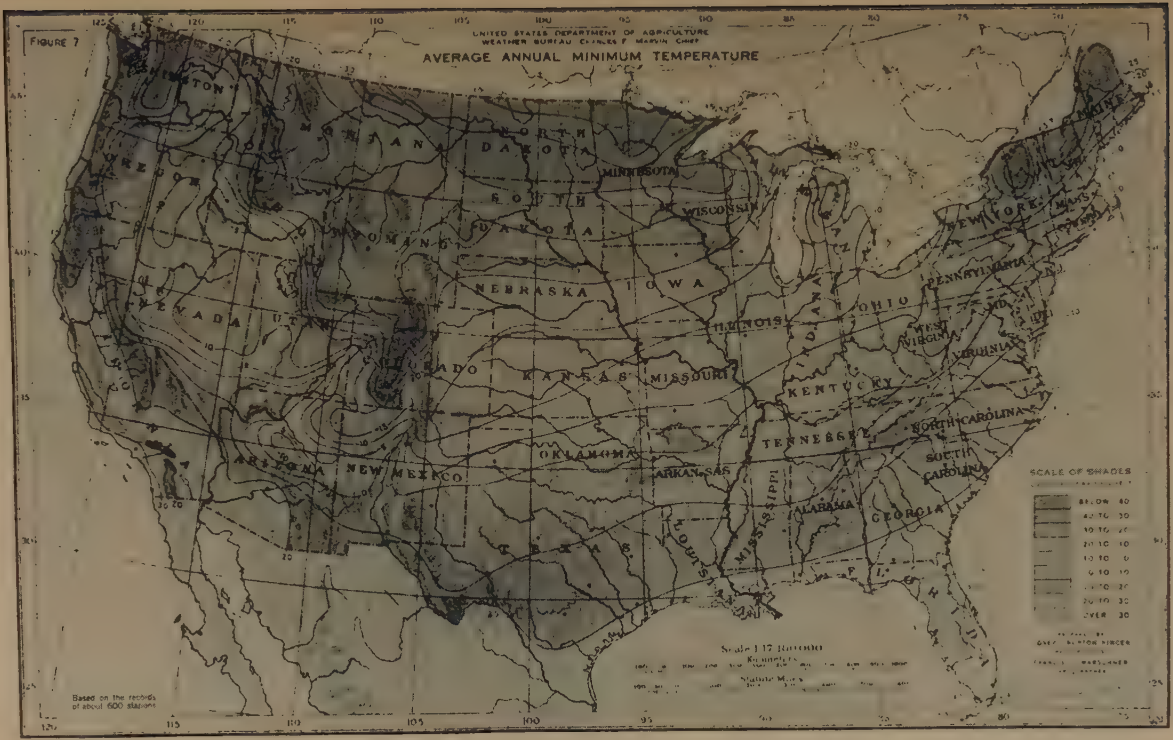


FIGURE 7
UNITED STATES DEPARTMENT OF AGRICULTURE
WEATHER BUREAU CHARLES F. MARVIN CHIEF
AVERAGE ANNUAL MINIMUM TEMPERATURE
WASHINGTON
OREGON
MONTANA
NORTH DAKOTA
MINNESOTA
SOUTH DAKOTA
WISCONSIN
WYOMING
NEBRASKA
IOWA
NEVADA
UTAH
COLORADO
KANSAS
MISSOURI
ILLINOIS
INDIANA
OHIO
PENNSYLVANIA
NEW YORK
MASS.
MAINE
MD.
WEST VIRGINIA
VIRGINIA
KENTUCKY
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
OKLAHOMA
ARKANSAS
ARIZONA
NEW MEXICO
TEXAS
LOUISIANA
MISSISSIPPI
ALABAMA
GEORGIA
FLA.
SCALE OF SHADES
BELOW 40
40 TO 30
30 TO 20
20 TO 10
10 TO 0
0 TO 10
10 TO 20
20 TO 30
OVER 30
Scale 1:17,000,000
Kilometers
Statute Miles
Based on the records of about 600 stations
125
120
115
110
105
100
95
90
85
80
75
70

Figure 5. This Map Shows the Average of the Lowest Temperatures Recorded Each Winter. As a Rule, the Lowest Temperatures in the United States Occur in the Northern Portions of North Dakota and Minnesota, Usually About — 40° or Slightly Lower. The Other Extreme is Found at Key West, Fla., Where the Lowest Temperature for the Year Ordinarily Does Not Go Below 50°. Along the Immediate Gulf Coast the Average Annual Minimum is 22° to 25°, Whereas Along the Immediate Pacific Coast it Ranges from About 25° at the North to 36° at the South. The Marine Influence is Markedly Shown by the North and South Trend of the Isotherms Along the Pacific Coast, and is Noticeable Along the Atlantic Coast, Where the Isothermal Lines Trend in a Northeasterly Direction as They Approach the Ocean and Terminate at the Coast Several Hundred Miles Farther North Than the Latitude at Which They Cross the Mississippi Valley. The Tempering Effect of the Great Lakes is Shown by the Trend of the Isotherms Along Their Leeward Shores in Michigan, Ohio, and New York

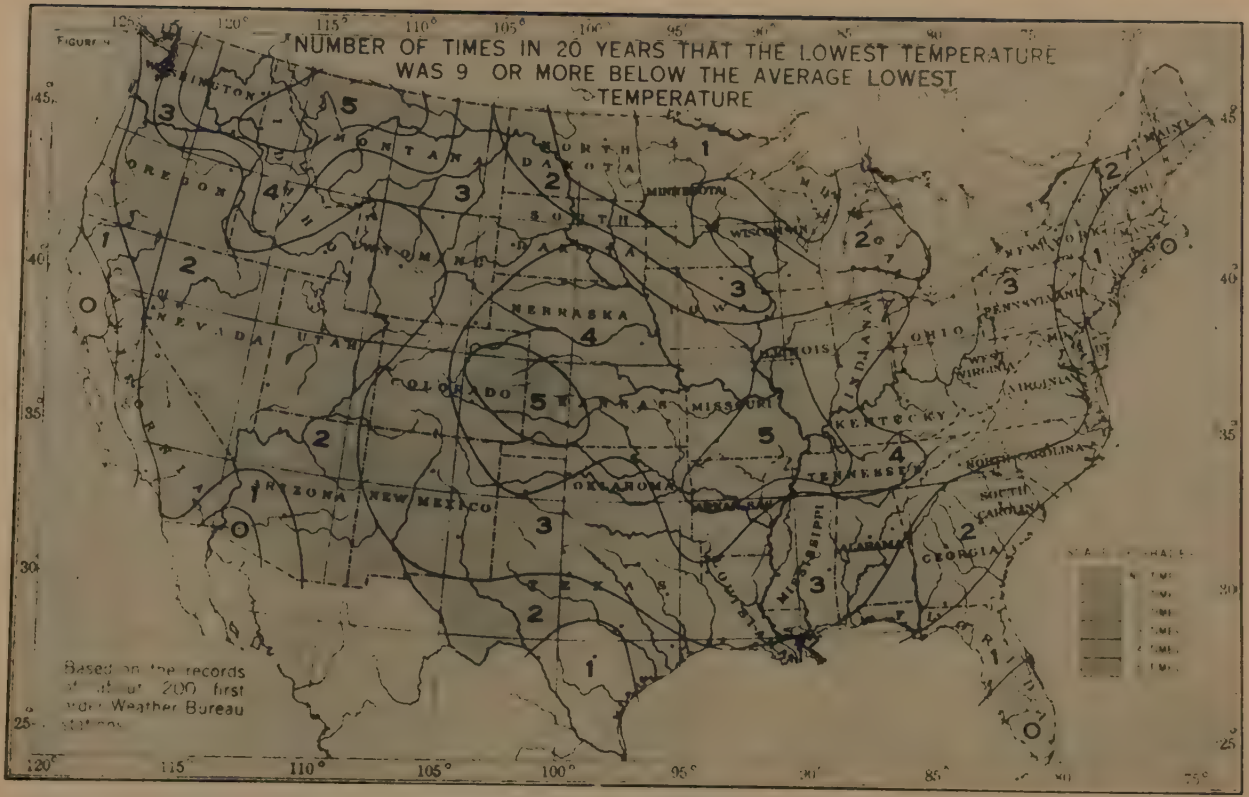

NUMBER OF TIMES IN 20 YEARS THAT THE LOWEST TEMPERATURE WAS 9 OR MORE BELOW THE AVERAGE LOWEST TEMPERATURE
Based on the records
Weather Bureau
125°
120°
115°
110°
105°
100°
95°
85°
75°
45°
40°
35°
30°
25°
WASHINGTON
OREGON
CALIFORNIA
NEVADA
UTAH
ARIZONA
MONTANA
WYOMING
COLORADO
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
MAINE
0
1
2
3
4
5

Figure 6. Number of Times in 20 Years that the Lowest Temperature was 9 Degrees or More Below the Average Lowest Temperature

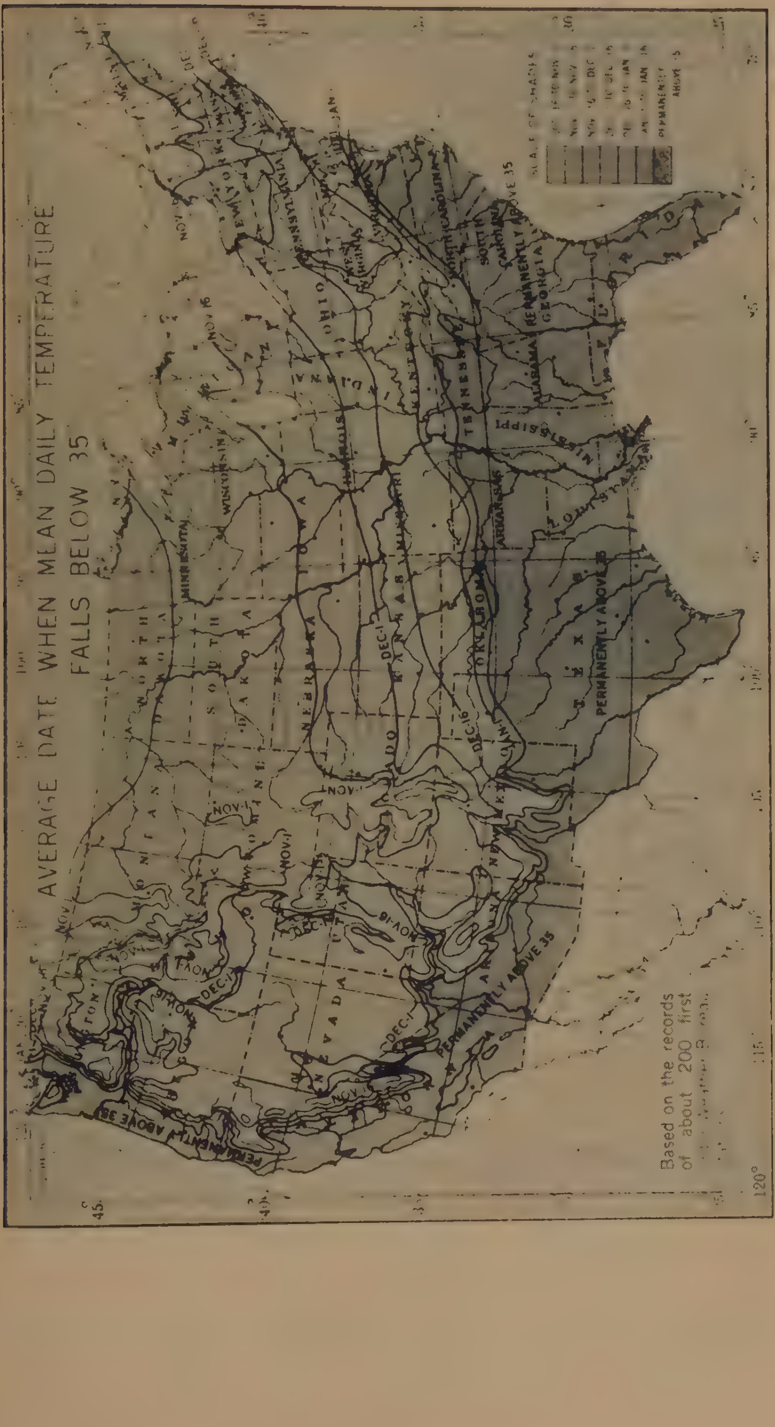

AVERAGE DATE WHEN MEAN DAILY TEMPERATURE
FALLS BELOW 35
SCALE OF SHADES
PERMANENTLY ABOVE 35
NOV. 1
NOV. 16
DEC. 1
DEC. 16
JAN. 1
PERMANENTLY ABOVE 35
Based on the records
of about 200 first
45°
40°
35°
30°
25°
120°
115°
105°
100°
WASHINGTON
OREGON
CALIFORNIA
NEVADA
IDAHO
MONTANA
UTAH
ARIZONA
NEW MEXICO
COLORADO
WYOMING
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MISSISSIPPI
MICHIGAN
INDIANA
KENTUCKY
TENNESSEE
ALABAMA
OHIO
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK

Figure 7. Average Date When Mean Daily Temperature Falls Below 35°

The State Highway Departments ought to be able to aid in collecting such information and possibly much valuable information might be obtained through the help of farmers.

Figure 5 shows the average annual minimum temperature throughout the United States.

In Figure 6, the average annual number of days in which the temperature is continuously below freezing is shown.

A study should be made of various frost phenomena, effect of the water content of the soil at the time of first freezing, the change in water content at varying intervals during the frozen period, the determination of alleviating agents such as snow covering, sod covering, black surfaces, cinder covers, black earth cover, depth of frost beneath the road, on the shoulder and in the field opposite, etc.

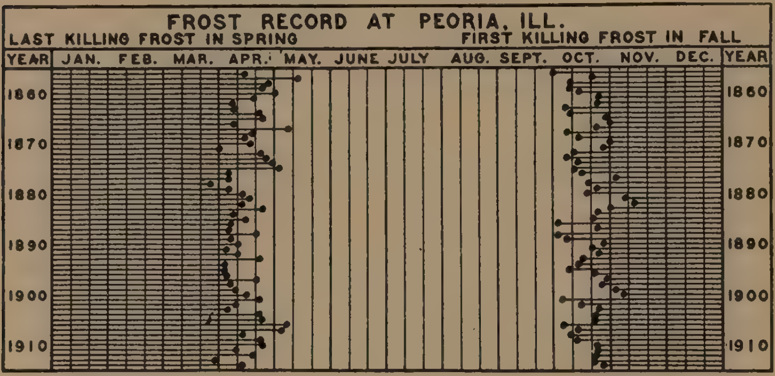


Figure 8. Frost Record at Peoria, Illinois

Figure 7 presents the data lines for the average date when the mean daily temperature falls below 35° F. This is a time when engineer and contractor must begin to take account of the placing of concrete in cold weather. Discussion in the committee brought out the fact that a temperature map for 40 or 42° F. would be better.

A similar map is shown in the folios for the time when the temperature rises above 35° F. in the spring.

To illustrate the fickleness of fall and spring, Figure 8 is presented showing the first date of killing frost in the fall and the last date of its appearance in the spring for a period of 60 years at Peoria, Illinois.

The average space between these events is the period of working time given to the concrete contractor, within which period he may quite safely operate. This was also attacked in the committee discussion as being too low a temperature for safety in laying concrete and that consequently, the working period should be much reduced.

Figure 9 gives the average date lines across the United States of the last killing frost in the spring as an average for a period of 20 years, from 1895 to 1914. A similar map is given of the first frost in the fall. Figure 10.

SUNSHINE AND WIND

The length of days for the June and December solstices are graphed for 5 degree intervals of latitude across the United States from Florida to Minnesota.

The length of day on December 22 varies from 8 hours and 26 minutes for northern Minnesota and Maine to 10 hours and 24 minutes for southern Texas and Florida.

The length of day from sunrise to sunset on June 21 ranges from 13 hours and 53 minutes for lower Texas and Florida to 16 hours for northern Maine and Minnesota.

Figure 11 shows the percentage lines of *actual* summer sunshine to the total possible sunshine at various points throughout the United States.

Figure 12 gives the lines of wind velocity throughout the U. S. at 3 p. m. when the wind velocity is normally at its greatest.

Abnormal winds such as Chinooks, blizzards, hot winds, low and high barometric-caused winds are also discussed in the text. It is the normal winds, that have the greatest effect upon climate because they carry the great amount of moisture laden air across the country.

Other graphs of sunshine and wind are given.

HUMIDITY

The humidity of the air affects road work in several ways. The setting and curing of concrete, the drying out of the subgrade, the curing of oil and tar treated surfaces. The amount of work done by the workmen, the efficiency of the work they do, all are affected by the amount of humidity in the atmosphere.

Humidity records are of two types, absolute humidity and relative humidity.

Absolute humidity is the actual amount of moisture held in the atmosphere, and is usually expressed in inches of barometric pressure, usually termed "vapor pressure." Absolute humidity depends upon the temperature and the proximity to large bodies of water.

Relative humidity is more important climatologically than absolute humidity. It is the ratio that the amount of moisture present in the

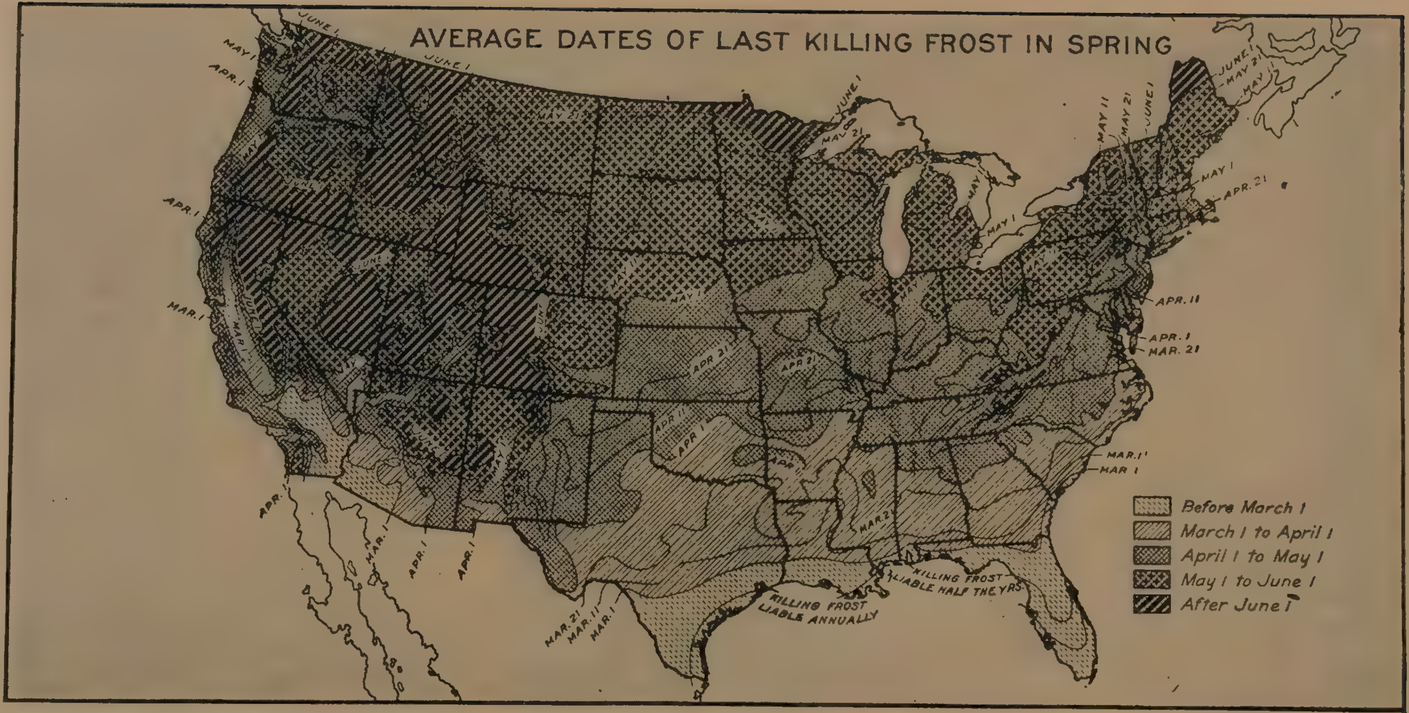

AVERAGE DATES OF LAST KILLING FROST IN SPRING
▨ Before March 1
▨ March 1 to April 1
▨ April 1 to May 1
▨ May 1 to June 1
▨ After June 1
KILLING FROST LIABLE HALF THE YRS
KILLING FROST LIABLE ANNUALLY
JUNE 1
MAY 21
MAY 11
MAY 1
APR. 21
APR. 11
APR. 1
MAR. 21
MAR. 11
MAR. 1

Figure 9. Average Date of Last Killing Frost in Spring. In Half the Years it Will Occur Earlier Than the Dates Shown and in Half the Years Later Than These Dates. It Follows That Spring Crops Which are Advanced Sufficiently to be Harmed by Frost on the Average Date of its Last Occurrence Will be Killed in 50 Per Cent of the Years. Killing Frost Usually Occurs Each Year to the Gulf Coast and to the Central Portion of the Florida Peninsula, but in Extreme Southern Florida it Occurs, as a Rule, in Less Than Half the Years. In Some Extreme Northern Districts and in the Higher Elevations of the Western States the Average Date of Last Killing Frost is Later Than June 1, but in the Gulf Coast Sections it is Infrequent After March 1

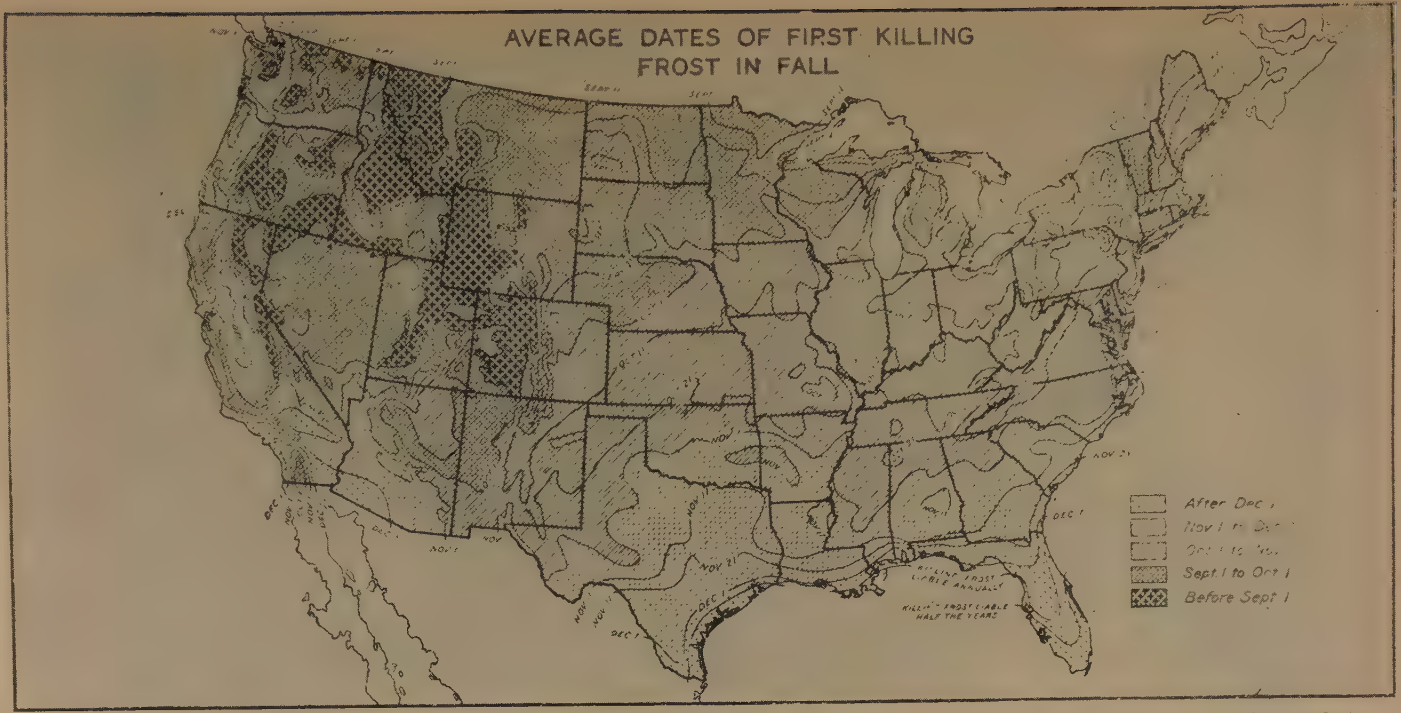

AVERAGE DATES OF FIRST KILLING FROST IN FALL
After Dec 1
Nov 1 to Dec 1
Oct 1 to Nov 1
Sept 1 to Oct 1
Before Sept 1
Killing frost liable annually
Killing frost liable half the years
Nov 21
Dec 1
Nov 1
Dec 1

Figure 10. Average Date of First Killing Frost in Fall, by Which Time it May be Expected to Occur in Half the Years. Many Summer Crops are Subjected to Frost Damage Before Maturity in Fall, Especially When Growing Conditions During the Summer are Such as to Retard Developments. The Average First Fall Frost Dates Range from Before September 1 in Some of the Colder Sections of the Country to December 1 on the East Gulf Coast. In Much of the Northern Portion of the Corn Belt the Growing Season Usually is Terminated by Killing Frost During the First 10 Days of October, Whereas in the Northern Cotton Belt the Average Time of First Killing Frost is Near the End of That Month. Killing Frost has Occurred at Every Point on the Mainland of the United States Where Records Have Been Kept. Key West, Fla., Being the Only Station with no Record of Killing Frost

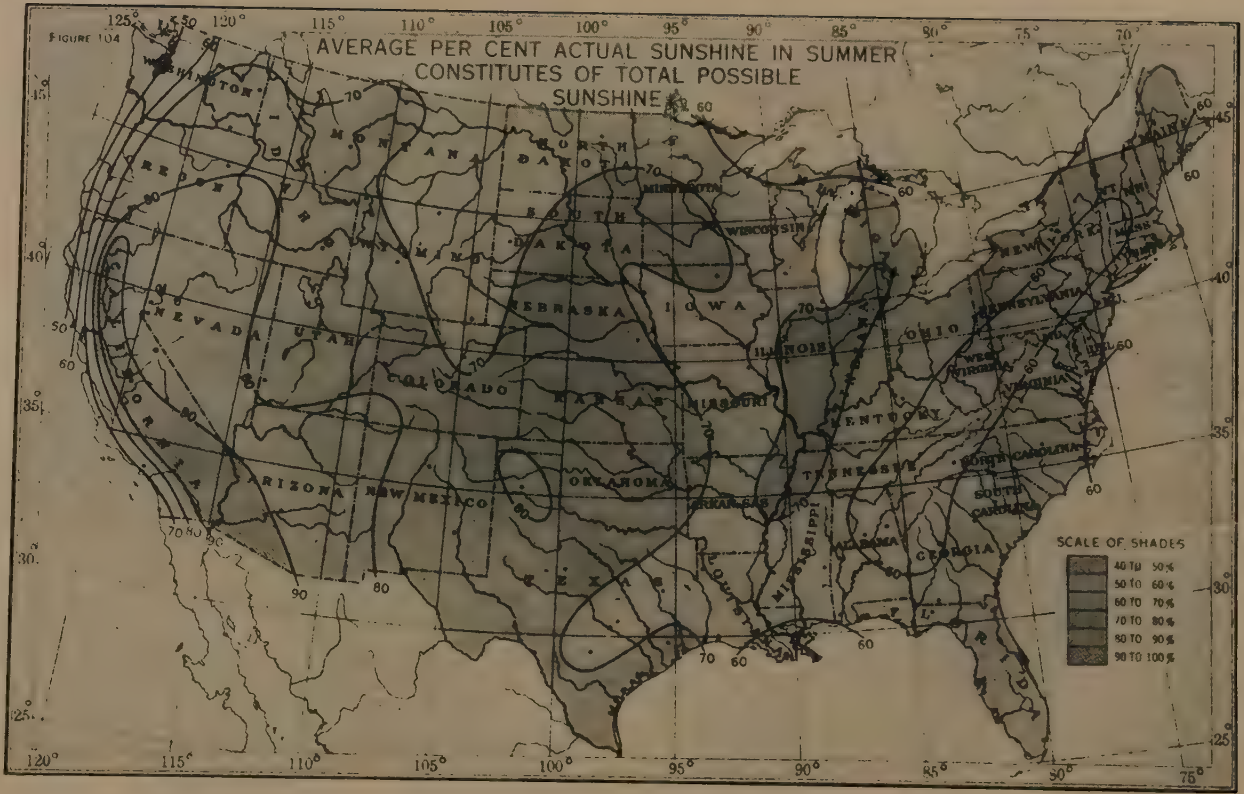


FIGURE 104
AVERAGE PER CENT ACTUAL SUNSHINE IN SUMMER CONSTITUTES OF TOTAL POSSIBLE SUNSHINE
SCALE OF SHADES
40 TO 50%
50 TO 60%
60 TO 70%
70 TO 80%
80 TO 90%
90 TO 100%
125°
120°
115°
110°
105°
100°
95°
90°
85°
80°
75°
70°
45°
40°
35°
30°
25°
WASHINGTON
OREGON
CALIFORNIA
IDAHO
NEVADA
MONTANA
WYOMING
UTAH
ARIZONA
COLORADO
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MISSISSIPPI
TENNESSEE
KENTUCKY
OHIO
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
MAINE
50
60
70
80
90

Figure 11. Average Per Cent Actual Sunshine in Summer

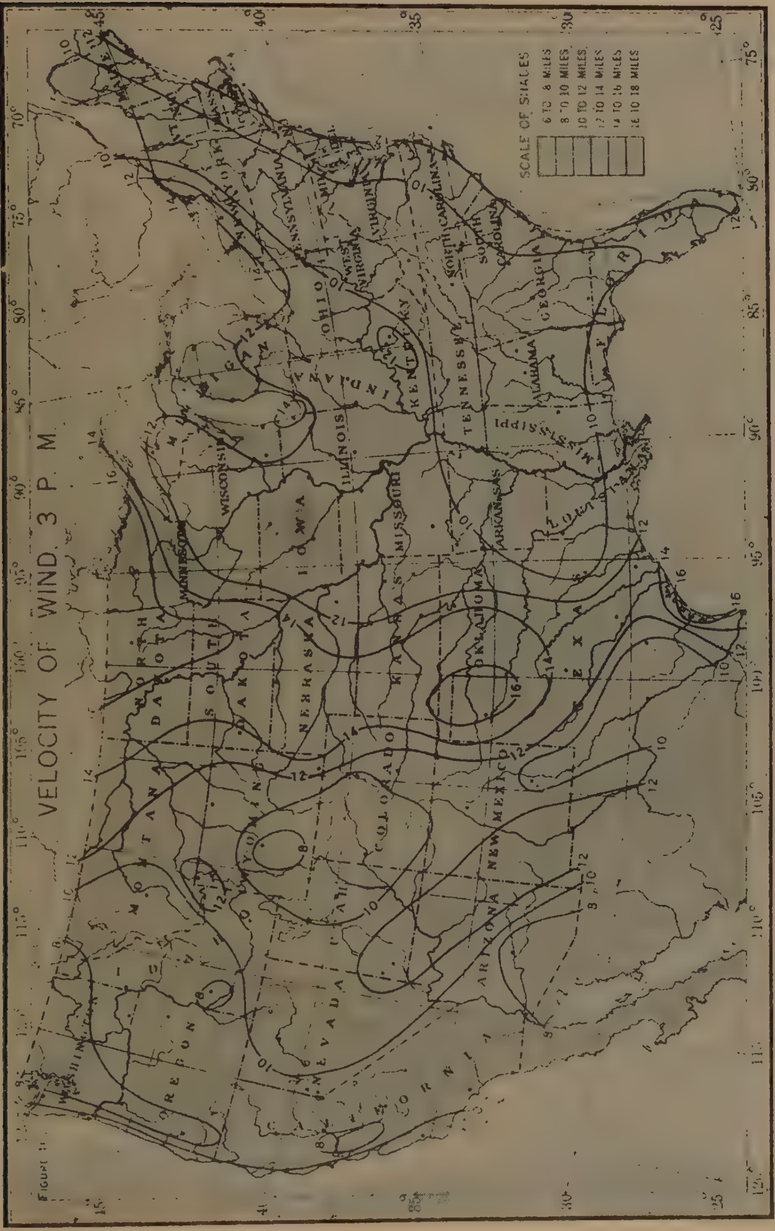

VELOCITY OF WIND. 3 P. M.
SCALE OF SHADES
6 TO 8 MILES
8 TO 10 MILES
10 TO 12 MILES
12 TO 14 MILES
14 TO 16 MILES
16 TO 18 MILES
FIGURE 11.
WASHINGTON
OREGON
CALIFORNIA
NEVADA
IDAHO
MONTANA
WYOMING
UTAH
COLORADO
ARIZONA
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
MAINE
MASS.
45°
40°
35°
30°
25°
70°
75°
80°
85°
90°
95°
100°
105°
110°
115°
120°
8
10
12
14
16

Figure 12. Average Velocity of Wind During the Year M. P. H. Figs. 106 and 107

air at any time bears to the amount that could exist there without condensation under the same conditions of temperature and atmospheric pressure.

Lower relative humidity means a higher evaporation rate and a higher humidity means a smaller rate of evaporation.

The dew point is the temperature at which saturation of the air occurs and the condensation of the invisible vapor in the air begins.

Figure 13 shows the humido bars for the average minimum relative humidity throughout the United States for October.

EVAPORATION

Evaporation is closely tied up with humidity and wind movement as well as with the temperature conditions. It varies largely during the year and throughout the United States. It occurs in the dead of winter from ice surfaces.

Some work has been done in obtaining data upon this subject but much needs yet to be done.

Figure 14 shows some of the data collected by various agencies. The evaporation is given in inches of water evaporated from water surfaces from April 1 to September 30. The figures enclosed in circles were observed by the office of the U. S. Biophysical Investigations, Bureau of Plant Industry, and cover periods of observation from 5 to 11 years in length.

The figures in the squares are data taken by the U. S. Weather Bureau during 1916-17.

The figures in the upright triangles are measurements made by the U. S. Weather Bureau, The U. S. Reclamation Service, and the U. S. Geological Survey during 1909.

These figures range from 25 inches along the Atlantic sea coast and the great lakes, to 88 inches in southern California and 93 inches in north central New Mexico.

Note that the greatest evaporation from water surfaces takes place at points where the least annual rainfall occurs.

PRECIPITATION

One of the very important if not the most important phases of climate relative to its effects upon the highway is the amount, distribution, intensity, character, and disposition of precipitation.

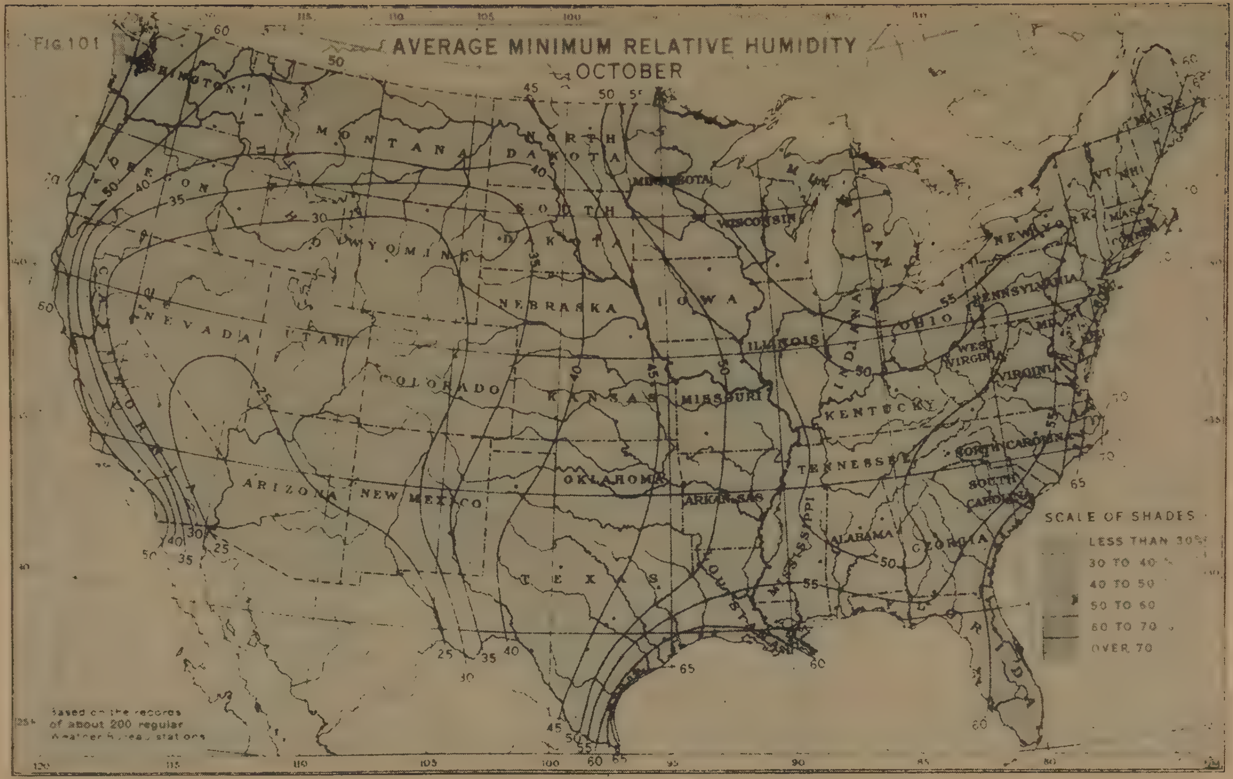


FIG 101
AVERAGE MINIMUM RELATIVE HUMIDITY
OCTOBER
SCALE OF SHADES
LESS THAN 30%
30 TO 40 %
40 TO 50
50 TO 60
60 TO 70
OVER 70
Based on the records of about 200 regular Weather Bureau stations
WASHINGTON
OREGON
CALIFORNIA
NEVADA
UTAH
ARIZONA
MONTANA
WYOMING
COLORADO
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
MAINE
VT
NH
MASS
CONN
MD

Figure 13. Average Minimum Relative Humidity, October

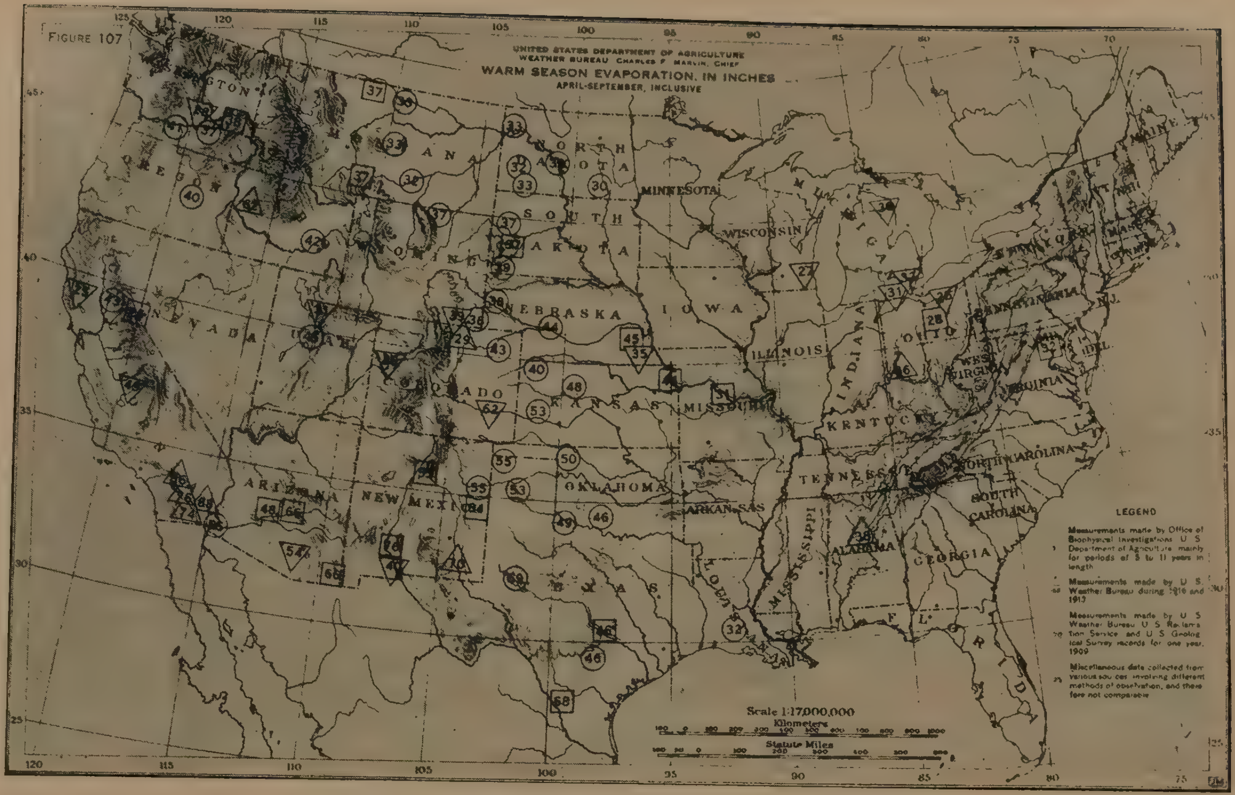


FIGURE 107
UNITED STATES DEPARTMENT OF AGRICULTURE
WEATHER BUREAU CHARLES F. MARVIN, CHIEF
WARM SEASON EVAPORATION, IN INCHES
APRIL-SEPTEMBER, INCLUSIVE
LEGEND
Measurements made by Office of Biophysical Investigations U. S. Department of Agriculture mainly for periods of 5 to 11 years in length
Measurements made by U. S. Weather Bureau during 1916 and 1917
Measurements made by U. S. Weather Bureau, U. S. Reclamation Service and U. S. Geological Survey records for one year, 1909
Miscellaneous data collected from various sources involving different methods of observation, and therefore not comparable
Scale 1:17,000,000
Kilometers
Statute Miles
WASHINGTON
OREGON
NEVADA
MONTANA
WYOMING
UTAH
COLORADO
ARIZONA
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
N.J.
DEL.
MAINE
37
30
33
32
30
33
37
37
33
42
40
40
48
53
62
50
55
55
53
49
46
45
35
44
43
38
29
31
27
28
32
46
46
68
54
68
66
48
72
120
115
110
105
100
95
90
85
80
75
70
45
40
35
30
25

Figure 14. Warm Season Evaporation, Fig. 107

It occurs in five forms, rain, snow, fog, hail, and dew. At least four of these produce problems that the engineer and contractor must meet and solve as economically as possible. All four affect the safety of the use of the road.

The U. S. Weather Bureau classifies rain, hail, and snow under the one head of precipitation.

Figure 15 shows the average annual precipitation in inches for the United States for a period of 20 years from 1895 to 1914, inclusive.

It varies from 5 inches in southern California and western Arizona to 60 or 70 inches in limited areas in the southeastern part of the United States and up to 120 inches in a limited area in western Washington.

The great Mississippi Valley ranges between 20 and 40 inches except for a small part in the southeastern section of the valley that rises to 50 and 55 inches per year.

In much of the great American desert precipitation falls below 10 inches annually.

This map is based upon the records of about 1600 stations for the entire 20-year period and upon about 2000 other stations running for shorter periods but adjusted to relate their data with the 20-year period.

Figure 16 indicates the variation in annual precipitation at selected stations over the United States, and it also illustrates the great difference in records between stations in different parts of the country.

Note especially the difference in precipitation between Winnemucca, Nevada, and Tullahoma, Tenn.

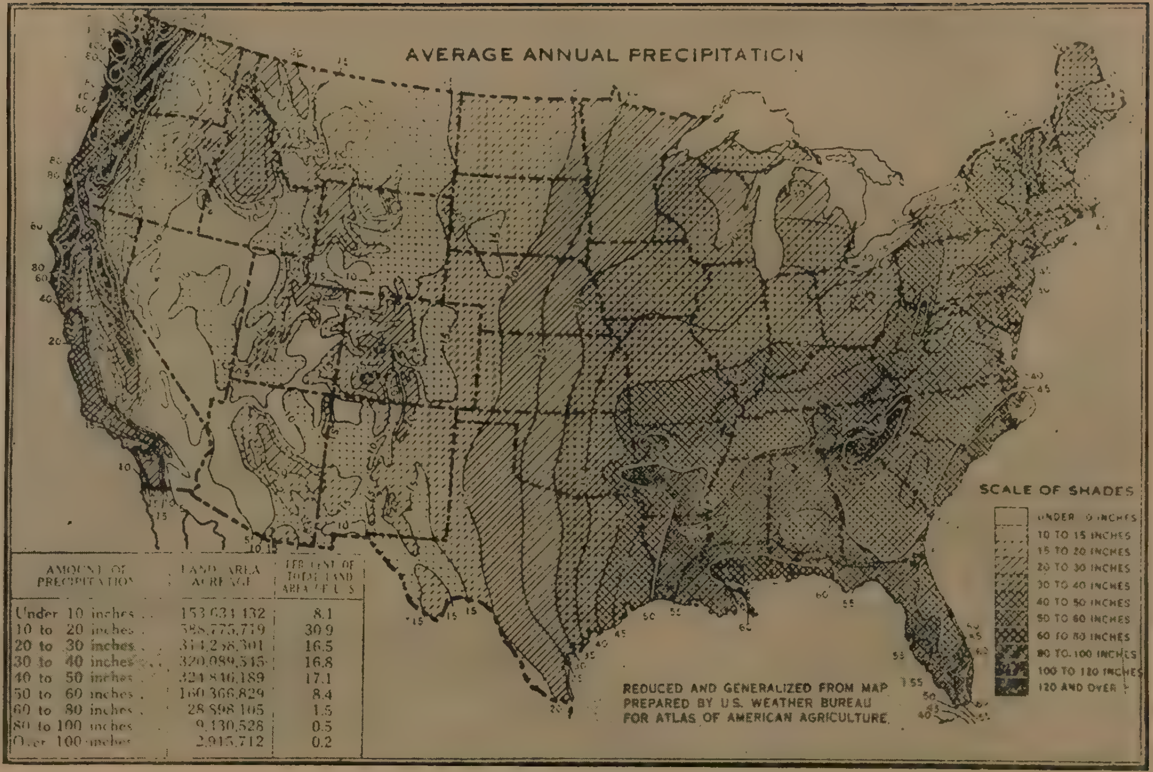
Figure 17 illustrates the fact that we have no records covering a sufficient period of time in this country to indicate any noticeable tendency toward permanent change in the rainfall statistics.

This figure shows the records from six long-record stations from different parts of the country.

The distribution of precipitation throughout the year is as marked a phase of climatic differences as is the difference of rainfall from different sections of the country.

Figure 18 shows this marked difference of rainfall types. It shows the percentage of annual rainfall which occurs each month at a number of selected stations throughout the country. The averages are again for the same 20-year period from 1895 to 1914.

Figure 19 shows a comparison of the average monthly precipitation for three 20-year periods taken in three cities. Here again no marked trend of change in amount can be noted.


AVERAGE ANNUAL PRECIPITATION
SCALE OF SHADES
UNDER 10 INCHES
10 TO 15 INCHES
15 TO 20 INCHES
20 TO 30 INCHES
30 TO 40 INCHES
40 TO 50 INCHES
50 TO 60 INCHES
60 TO 80 INCHES
80 TO 100 INCHES
100 TO 120 INCHES
120 AND OVER
REDUCED AND GENERALIZED FROM MAP
PREPARED BY U.S. WEATHER BUREAU
FOR ATLAS OF AMERICAN AGRICULTURE.

AMOUNT OF PRECIPITATION	LAND AREA ACREAGE	PER CENT OF TOTAL LAND AREA OF U. S.
Under 10 inches	153,631,132	8.1
10 to 20 inches	588,775,719	30.9
20 to 30 inches	314,258,301	16.5
30 to 40 inches	320,089,545	16.8
40 to 50 inches	324,846,189	17.1
50 to 60 inches	160,366,829	8.4
60 to 80 inches	28,598,105	1.5
80 to 100 inches	9,130,528	0.5
Over 100 inches	2,915,712	0.2

Figure 15. Average Annual Precipitation

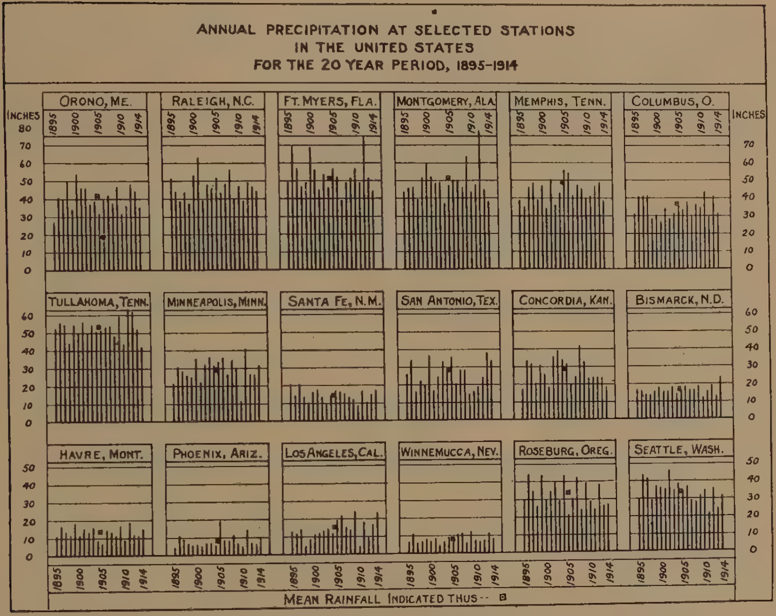


Figure 16. Annual Precipitation at Selected Stations for the 20-Year Period, 1895-1914, Arranged by Geographic Districts

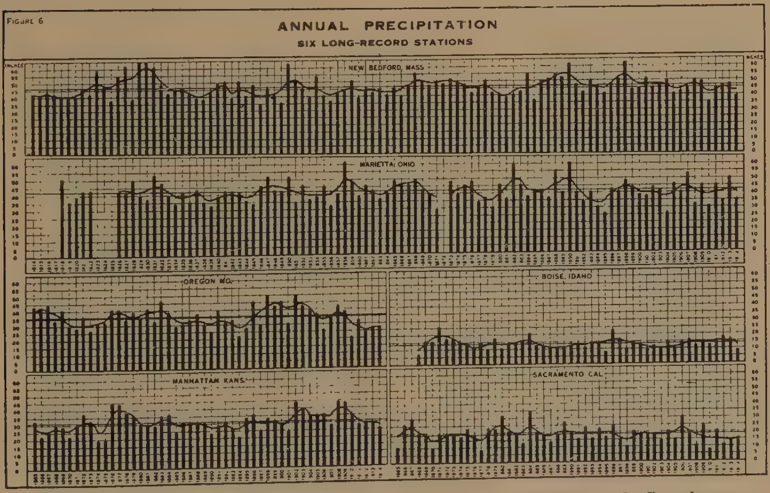


Figure 17. Annual Precipitation. Six Long-Record Stations

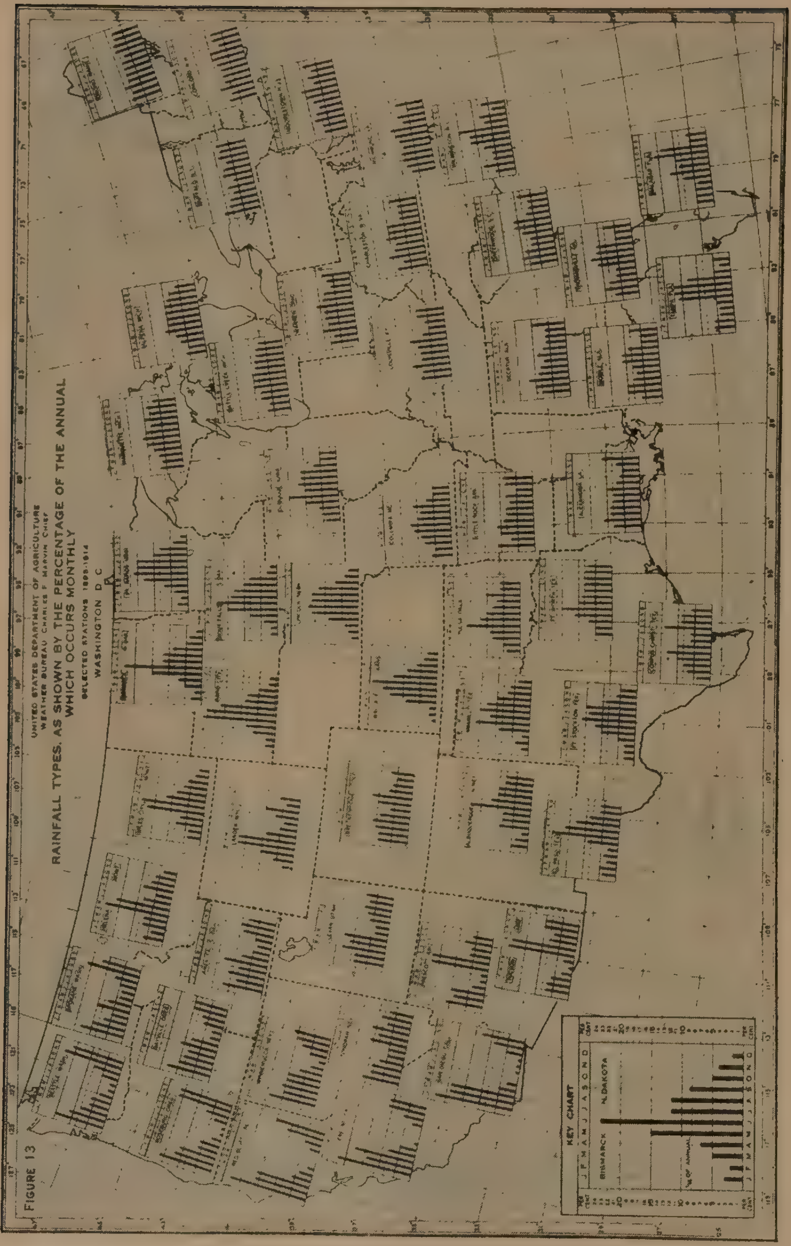

UNITED STATES DEPARTMENT OF AGRICULTURE
WEATHER BUREAU CHARLES F MARVIN CHIEF
RAINFALL TYPES, AS SHOWN BY THE PERCENTAGE OF THE ANNUAL WHICH OCCURS MONTHLY
SELECTED STATIONS 1895-1914
WASHINGTON D C
FIGURE 13
KEY CHART
BISMARCK
N. DAKOTA
J F M A M J J A S O N D

Figure 18. Rainfall Types

Figure 20 shows the total number of periods of 20 consecutive days or more without 0.25 inch of rainfall occurring in 24 hours during the season from March 1 to September 30, for the 20-year period above used. This map only includes that portion of the United States east of the great plains.

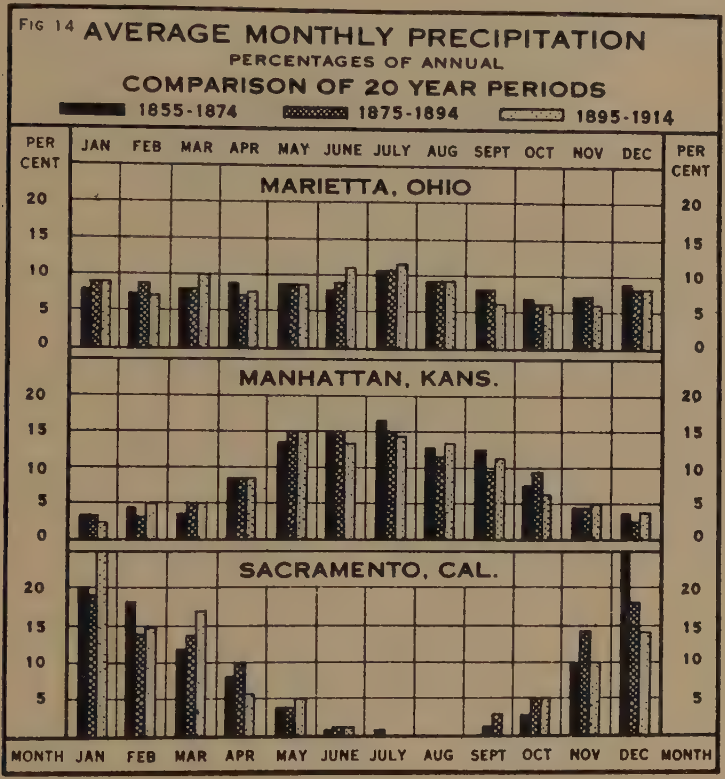


Figure 19. Average Percentages of the Annual Precipitation That Occurred in Each Month During Three Successive 20-Year Periods at Marietta, Ohio, Manhattan, Kans., and Sacramento, Cal., Representing the Three Principal Rainfall Types in the United States. Solid Bars Represent the Period, 1855-1874; Closely Shaded Center Bars, 1875-1894; and Lightly Shaded Bars at the Right, 1895-1914

Snowfall. The precipitation in the form of snowfall introduces another set of problems for the engineer and highway executive. It increases the dangers to travel and may totally prevent travel for considerable periods. It increases the cost of maintenance; may reduce the ill effect from frost; may increase the instability of the subbase by holding on and supplying to it a surplus amount of water to saturate and soften not only the subbase but the road shoulders and surrounding areas; and in general it becomes an expensive form of precipitation to handle.

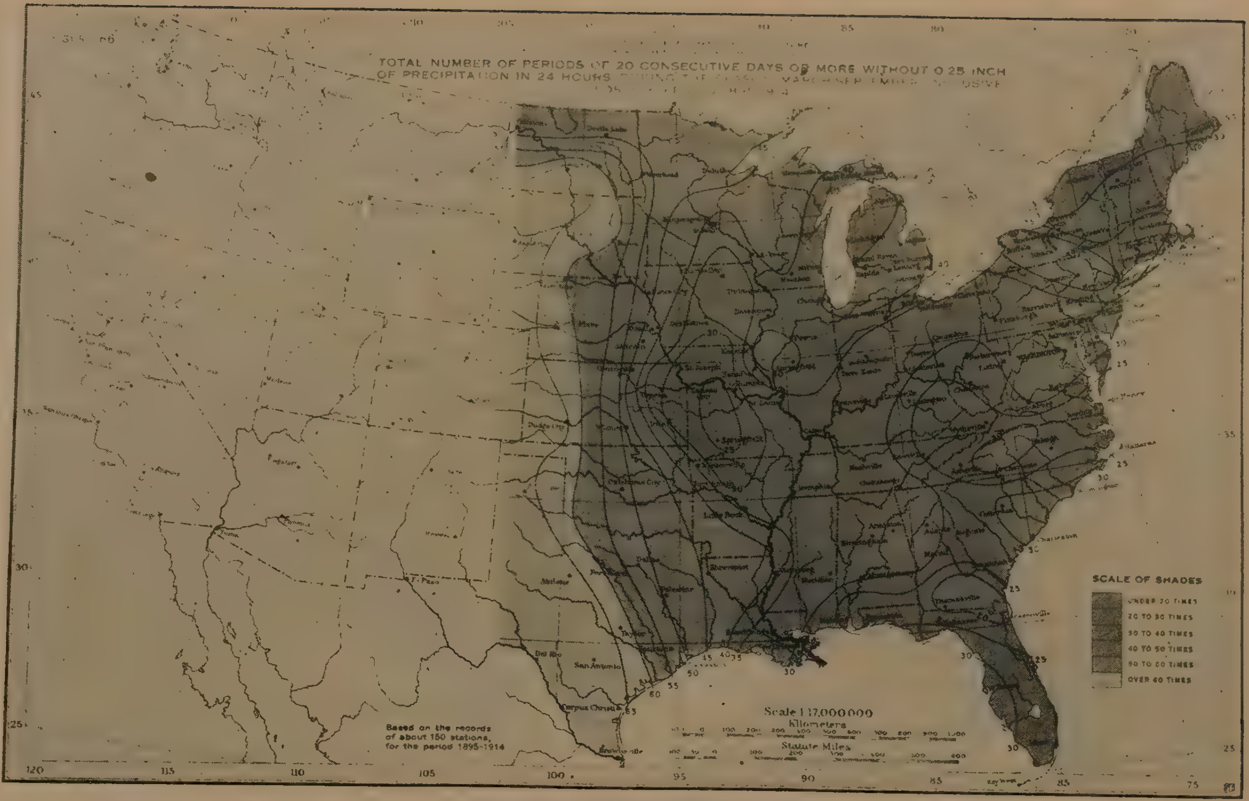

TOTAL NUMBER OF PERIODS OF 20 CONSECUTIVE DAYS OR MORE WITHOUT 0.25 INCH
OF PRECIPITATION IN 24 HOURS
SCALE OF SHADES
UNDER 20 TIMES
20 TO 30 TIMES
30 TO 40 TIMES
40 TO 50 TIMES
50 TO 60 TIMES
OVER 60 TIMES
Based on the records
of about 150 stations,
for the period 1895-1914
Scale 1:17,000,000
Kilometers
Statute Miles
San Antonio
Corpus Christi
El Paso
Little Rock
Shreveport
120
115
110
105
100
95
90
85
80
75
45
30
25

Figure 20. Total Number of Periods of 20 Consecutive Days or More Without 0.25 Inch of Precipitation in 24 Hours

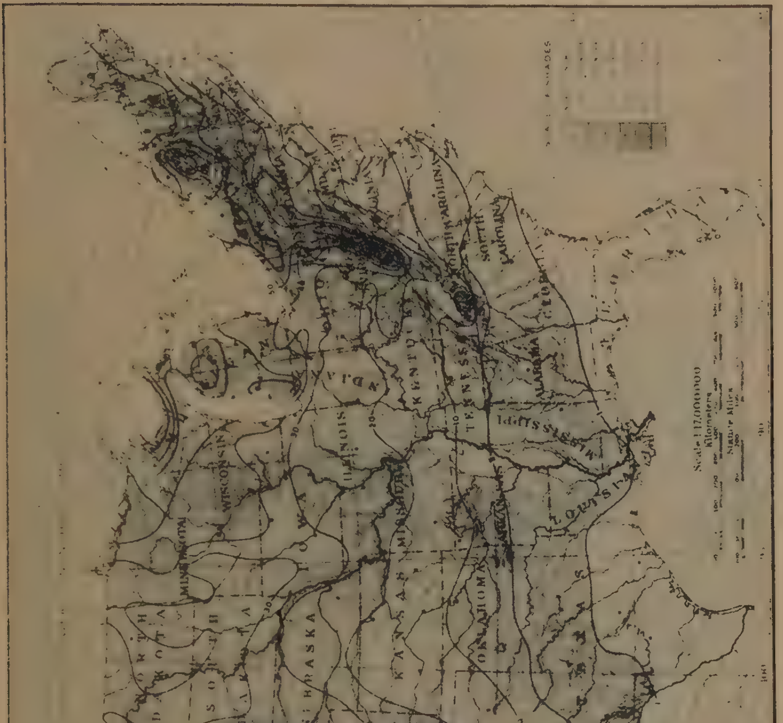

NORTH DAKOTA
SOUTH DAKOTA
MINNESOTA
WISCONSIN
IOWA
NEBRASKA
KANSAS
MISSOURI
ILLINOIS
INDIANA
OHIO
KENTUCKY
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA
GEORGIA
ALABAMA
MISSISSIPPI
ARKANSAS
LOUISIANA
OKLAHOMA
TEXAS
NEW MEXICO
Scale 1:17,000,000
Kilometers
Statute Miles

Figure 21. Average Annual Snowfall in Inches Unmelted

Figure 21 shows the lines of average annual snowfall in inches of unmelted snow, throughout the country.

The greatest annual snowfall occurs on the western slopes of the Sierra Nevada and Cascade mountain ranges where in some places it averages from 400 to 500 inches per year.

East of the Rocky Mountains the greatest snowfall is in a small area along the southern shore of Lake Superior where 120 inches per year is recorded.

Figure 22 shows the average annual number of days with snow cover on the ground in the United States.

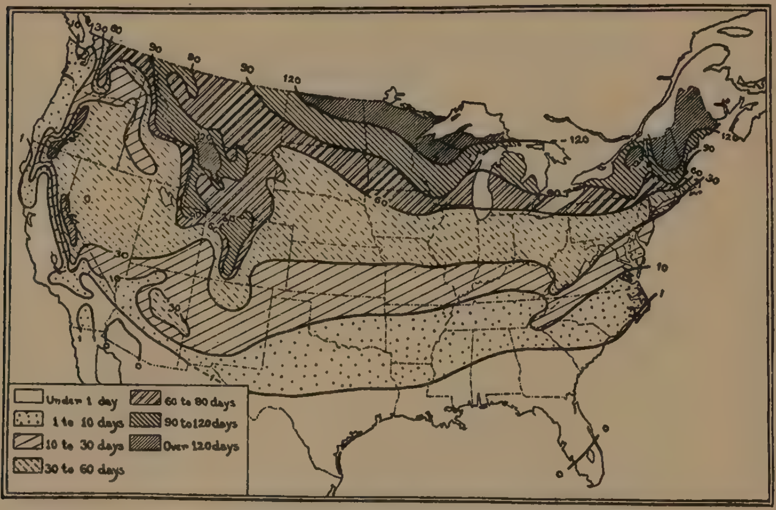


Figure 22. Average Annual Number of Days With Snow Cover

Hail occurs on an average of but two days per year over the greater portion of the United States. Over a small area in the Rocky Mountains it averages 4 days per year for the frostless season.

The U. S. Weather Bureau recognizes two degrees of fog—dense and light. If objects are obscured to the sight at 1000 feet it is classed as a dense fog.

The Pacific Coast is the most afflicted area in the country. A strip along the sea coast running back to the Sierra Nevada mountains is affected from 40 days per year near the coast to 10 days per year back on the mountain ridges.

The great American plains have 5 days or less of dense fog per year. The eastern half of the United States seldom exceeds 10 days of dense fog per year except for small areas in northeastern New England states and along the tops of the Allegheny Mountains.

The danger from fogs are twofold—first, slippery roads; and second, occluded vision.

Figure 23 shows the annual number of clear days for the 20-year period from 1895 to 1914.

The range is from 80 days in western Washington, 100 days in northern Michigan, Ohio, Pennsylvania and New York, to 300 days in southeastern California and western Arizona.

RUNOFF

Runoff of rainfall is not usually considered when alluding to climate. It is so intimately connected, however, with every factor in climate that it is here included.

The relations existing between precipitation, percolation, evaporation, surface retention and runoff are so varied and so largely affected by the distribution, intensity, duration, amount, and character of precipitation as well as by temperature, wind and humidity, that formulas and theories can but imperfectly apply.

The best that formulas can do to express the probable amount of run-off for any given area is to approach some approximation. Long continued measurements give what has occurred and point approximately to what may occur under similar conditions in the future.

The comparison of the runoff from one area with what may occur on another area apparently similarly located may be quite distinctly in error, depending largely upon the question as to how many of the variables that enter into each result are of similar value.

The United States Geological Survey has been measuring the runoff in streams in the United States since 1888. For many years this was done at only a few points upon a few of the streams. At present there are 2002 active stations maintained in the United States and Hawaii, while 2562 have been discontinued. It is estimated that about 4500 stations would be required for a complete inventory of stream flow.

The records from the 2002 active stations represent a total of 17,569 years of observation.

The results of stream-flow measurements are now published annually in 12 parts, each part covering an area whose boundaries coincide with the natural water shed lines of some great drainage basin.

Complete sets of the publications of the U. S. G. S. may be consulted at 29 widely distributed points in the United States and at Honolulu, Hawaii.

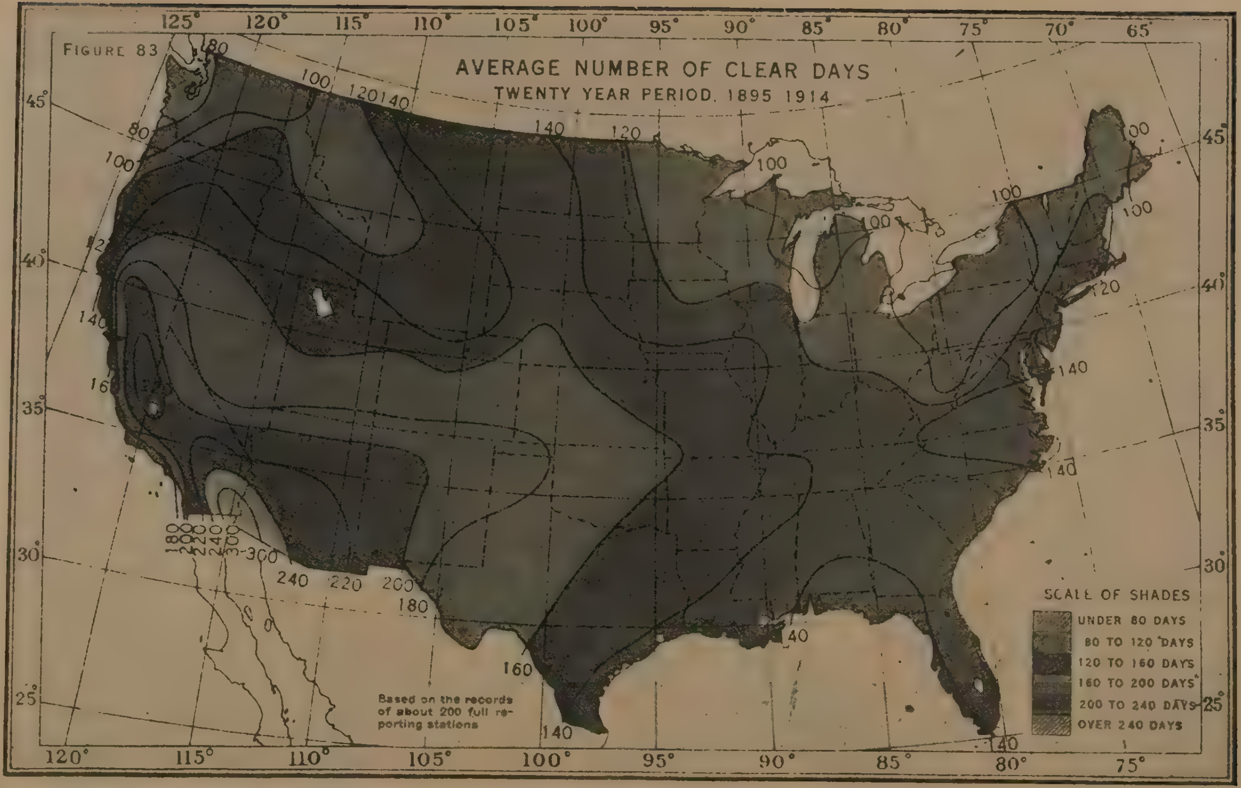


FIGURE 83
AVERAGE NUMBER OF CLEAR DAYS
TWENTY YEAR PERIOD, 1895 1914
SCALE OF SHADES
UNDER 80 DAYS
80 TO 120 DAYS
120 TO 160 DAYS
160 TO 200 DAYS
200 TO 240 DAYS
OVER 240 DAYS
Based on the records of about 200 full reporting stations
125° 120° 115° 110° 105° 100° 95° 90° 85° 80° 75° 70° 65°
45° 40° 35° 30° 25°
80 100 120 140 160 180 200 220 240 300

Figure 23. The Annual Number of Clear Days in the Twenty-Year Period from 1895 to 1914. Cloudy Weather Occurs With Greatest Frequency Along the North Pacific Coast in the Region of the Great Lakes Where There Are on an Average, Fewer Than One Hundred Clear Days a Year. In Portions of the Southwest, Three Hundred or More Days are Clear

The papers themselves may be secured at a nominal cost from the Superintendent of Documents at the Government Printing Office, Washington, D. C.

Another source of valuable information is contained in the large number of papers published in the Transactions of the American Society of Civil Engineers from 1900 on to the present time.

Some eighteen or twenty papers not including nearly as many more published in connection with the Mississippi flood Symposium, discuss every angle of rainfall, runoff, and stream flow.

In this preliminary report there is not sufficient time to enter further into the question of runoff.

An editorial in Engineering News-Record of October 17, 1929, calls attention to the comparison of bridge losses in two southern states caused by the heavy storm of September 30 to October 3, 1929, which was about equally severe in each state. In one state not a bridge built by the State Highway Department was lost. In the other, 13 wooden and 5 concrete structures suffered. Quoting, the editorial says, " The comparison between the flood experiences of the two states serves to show the value of an intensive engineering study of bridge hydraulics, and the adequate public support for enduring bridges."

THE APPLICATION OF CLIMATOLOGICAL DATA

The question arises, Is there sufficient need for information along the lines just presented to warrant the time of a committee spent upon the work? Of what practical value will such information be to the road builder and road user?

The committee has attempted to indicate briefly the value as each item has been presented.

It will be well, however, to summarize some of the more practical uses for this information, and for further information that should be collected.

The engineer must know how large to make his bridge and culvert openings. The losses sustained in South Carolina contrasted with the insignificant losses sustained by North Carolina from the early October storm emphasize this need.

In order to be prepared for every phase of drainage that arises for the highway engineer, he must be able to put his fingers on the best data available regarding precipitation in all forms, runoff, percolation, stream flow and stream velocity, surface retention of water, evaporation, the effects of humidity, wind, temperature, sunshine and frost as they unite in affecting the runoff and his subdrainage problems.

The stability of the subgrade soil is dependent upon its freedom from saturation which is affected by rainfall, snow, frost, evaporation, temperature, wind, sunshine and the humidity of the air above it.

The action of frost in the soil and cold weather above the soil upon capillary water, the freezing of free water in clear layers of ice in the soil beneath the pavement, due, as Professor Taber says, to the molecular cohesion, form problems that the engineer must be prepared to meet.

The contractor knows only too well the effect of both extreme heat and extreme cold upon the *amount* of work performed by his forces and the greater frequency with which he discovers defects in their work. For these reasons he must make allowance in his estimates and bids.

The other elements of climate also act in a similar manner but possibly with less effect.

The maintenance of the road shoulders and ditches are largely affected by various elements of climate. The maintenance engineer must organize his forces for different classes of work as the various elements of climate alternate in their attacks.

Tar and asphalt surfaces may be softened by heat and become badly deformed under traffic, or under excessive cold become brittle and crack, admitting water beneath the surface to injure and destroy the foundation and subgrade.

Pavements may be heaved up or displaced by the swelling or shrinking soil or by frost so that they become shattered or totally destroyed.

Snow brings up the organization of the maintenance forces for the setting of snow fences, snow plowing, cinder covering of hill slopes, and the opening of ditches and drainage channels in the early spring.

The engineer may consider or neglect certain construction and maintenance features if he knows the average, maximum, and minimum limits of these various climatic factors.

The user of highways is met at some definite period in the spring with a sign: " Loads limited to (5 or) 7½ tons until further notice." The slipperiness of many roads is caused by fog, rain, ice, snow, and mud from side roads. The length of vision and the safety in use of road is affected by sun, fog, rain, snow, and wind.

NEEDED RESEARCH

1. The depth to which frost penetrates the soil in various localities over the country should be determined.
2. The relation of frost depth to the length and the severity of the cold weather season or spell should be sought.
3. The influence of various covers to prevent deep freezing should be tried out. (Especially covers of adjacent road shoulders.)
4. Runoff data has been widely taken, but a condensed volume of the best data should be compiled and a concise statement regarding the various formulas made.
5. Further research into the field of evaporation and of percolation should be made.
6. The actual effect in reduced labor efficiency, and number of defects caused by the influence of these various factors upon labor, should be studied.
7. Determination of the best means of determining the field percolation value of any given soil, etc.

DISCUSSION

ON

THE INFLUENCE OF CLIMATE ON THE BUILDING, MAINTENANCE AND USE OF ROADS IN THE UNITED STATES

Mr. J. A. Sourwine, *U. S. Bureau of Public Roads.* Mr. Eno's paper presents a phase of research, which as a general study is not new, but which as applied to highway design and construction opens new and interesting fields of study.

I wish to discuss two items of the paper:

(1) Ground freezing.
(2) Precipitation and runoff.

Perhaps in any phase of study it is worth while to get a general picture of the relative value, and dollars and cents importance, of the general project. A study of the damage to State Highways during the winter before last (1927-8), was made by the Engineering News-Record, and approximate estimates of frost damage were submitted by various state highway organizations. Minnesota reported 2000 miles of highway affected by ground freezing, and stated that the loss by reason of interrupted traffic and delayed business would be still greater than the direct damage. Michigan reported an average frost damage of $50 per mile on gravel, $100 per mile on macadam and $150 per mile on concrete pavements. Ohio estimated frost damage at one-quarter million dollars. Maine reported an estimated cost of one-half million for required repairs. Lacking complete reports from all states, but taking the reports available, and supplying an approximate estimate where state highway reports are not available, the amount of state highway repairs required, covering frost damage during the winter season 1927-8, is approximately three million dollars. This estimate covers state highways only. If we apply an arbitrary estimated additional percentage in the ratio of two-thirds of the state highway repair cost, to cover allowance for frost damage to municipal and county roads, we obtain an estimated total highway frost damage in the United States, for the winter of 1927-8, of five million dollars. Such general inquiry as I have been able to make, indicates that the frost in the winter 1928-9, was not widely different, on the whole, from the winter 1927-8. Capitalizing this damage at five per cent, we have an approximate estimated value of the highway ground-freezing problem in the United States. Twenty-nine states are directly affected and over two-thirds the population of the United States. This gives us a general picture of the relative magnitude of our problem.

When we turn to consider in detail the ground-freezing problem, we encounter: First, the problem of freezing weather during construction, Second, the problem of low temperature occurrence, duration and frequency, over a longer period, being the estimated life period of the highway under consideration. I will limit my discussion to the latter, being the phase of study affecting highway design.

The first question we face is—can existing records of past climatic conditions be used as a guide, to assist us in determining what is relatively dangerous area, and what is relatively safe area, for highway ground-freezing? Certainly the records of absolute minimum temperatures may not be so used. An illustration is the case of Washington, D. C. The absolute minimum temperature which has ever occurred here is — 15° F. A large number of those present are reasonably familiar with Washington. It is probably safe to say that common opinion would not concur in accepting the extreme low minimum of — 15° F., as in any way typical, or as giving any basis which may be used, for comparative study, as representative of Washington,

D. C. Because this low temperature has occurred once, during an available period of record of 48 years, does not make this value a basis for design.

After some study of United States weather data, the lowest monthly average of daily minimum temperatures—during winter months—was selected as offering a basis for comparative study and for serving as a guide, in determining approximate relatively dangerous and relatively safe areas, from the low temperature standpoint, as affecting highway ground-freezing. The detail study leading to the determination of a tentative critical value, for lowest average of daily minimum, will be presented later in written form through report to the U. S. Bureau of Public Roads. This much of general discussion may be given. Using the lowest monthly average of daily minimum temperatures, an isothermal chart has been prepared—first, one chart for each state through which the tentative critical isothermal passes, and then one combined chart, to a smaller scale, for the entire United States. Some data as to low winter rainfall, indicating arid regions, where the relative danger of highway ground-freezing seems more or less doubtful, at moderate extremes of cold temperature, is shown also on the same chart. This chart is not calculated for use as an arbitrary determinator of areas dangerous for ground-freezing—but it is submitted as a guide chart, which, supplemented by other local data and special studies as available, will possibly form a tentative framework, on which may be built up, following several years of further study, and modification as required—an accurate reference chart, indicating climatic territory relatively safe, and relatively dangerous, for highway ground-freezing.

Let us go one step further. We have considered only low temperatures. How about the " duration of cold period," and how serious a modifying element is this item of duration? The best reply is to cite some actual field tests made by Mr. George J. Bouyoucos—at the Michigan Agricultural College Experiment Station. Throughout the month of January, 1915, Mr. Bouyoucos ran a series of comparative temperature measurements in air and in soil. His air temperatures were taken in air four feet above ground surface, and his soil temperatures at a depth of three inches below the surface of the ground. His records show daily minimum, daily mean, and daily maximum temperatures, in ground and in air. A cold period began on January 20, when the minimum air temperature dropped to 8° F. There followed a continuous cold period, for the remainder of the month. On the eighth day of this cold period the minimum air temperature dropped to — 6° F. On the ninth day the minimum air temperature was — 13° F. On the tenth day the minimum air temperature was — 13° F., or exactly the same as on the ninth day. The mean daily air temperatures for the three days, 28th to 30th inclusive, were 2° F., 0° F., and 2° F. In a word, the mean air temperature for the tenth day of the cold period was 2 degrees warmer than on the ninth day, and the minimum air temperature was the same. Yet the minimum soil temperature, at a depth of three inches below the surface, was, on the ninth day, 14.5° F., and on the tenth day 7.5° F. In a word, the minimum air temperature was unchanged, but the additional duration of extreme cold period caused the minimum soil temperature, at three-inch depth, to drop 7° F. The answer to our question is plain. The record answers for us. Duration of cold period is a factor we must consider, as affecting relative danger of ground-freezing, in any locality.

Duration, being a factor, we have seen three variables: (1) Minimum temperature, (2) duration of cold period, and (3) frequency of cold period. The question comes, can we in any way combine or relate two or more of these variables, in order that we may obtain a closer determination of territory relatively dangerous and relatively non-dangerous for highway ground-freezing, from the standpoint of climatic conditions? After some puzzling, the answer comes. We have combined variables through a simple method, in other engineering studies. It is only necessary to apply a similar principle here. We are all familiar with the term " ton-miles." Why should we not use with equal accuracy, and with similar degree of effectiveness, the term " degree-hours "? In such a combining of the " minimum temperature-cold period duration " factors, we have to determine first our critical temperature, at or below which ground-freezing begins, in average soil. The " degrees of cold " are then measured downwards in degrees below the critical temperature. The " duration " is measured in hours of cold period below critical temperature. One-half the product of " degrees of cold " and " duration," as above measured, gives " degree-hours," measuring the approximate actual value of " quantity of cold," in any given cold period.

Using the method just outlined, for any point where a continuous thermograph record is available, over a period of years, we may tabulate the total number of cold periods below critical temperature, with the duration and minimum temperature of each cold period. If from these tabulated data, we plot two curves, (1) a minimum temperature-frequency curve, and (2) a cold period duration-frequency curve; we can then, from these two plotted curves, determine and plot a third curve, a " degree-hour-frequency curve," which takes into account all three variables, of minimum temperature, cold period duration, and cold period frequency. It rests then with the executive body planning new highway improvement, to determine on what basis of allowable frequency they desire to base their highway design. For the allowable frequency fixed upon, the climatic records of actual previous occurrence determine a definite " degree-hour " value.

Following up the " degree-hour-frequency " method, or " quantity of cold " method, the thought is suggested that if we may agree upon a tentative value of " allowable frequency," for cold period occurrence, as a basis for design, we may determine and plot a series of " degree-hour " contours, in any state, or across the United States, which degree-hour contour chart may be used as a supplementary chart, to the temperature—rainfall chart already determined and plotted, and by the use of these two charts, we may have available a useful guide, for the determination of climatic territory relatively safe, and relatively dangerous, for highway ground-freezing. Such a " degree-hour contour chart " is now being determined and prepared, and will be made available in a report to be issued shortly by the U. S. Bureau of Public Roads.

Now if I may touch very briefly on the second item of Mr. Eno's paper, which I wish to discuss—" Precipitation and Runoff." Or possibly we may elaborate slightly Mr. Eno's heading, and say " Precipitation, Runoff, and Stream Control, as affecting public highways." The question I wish to call to your attention is—" What relative importance has this phase of study—and may it properly and directly be incorporated as a part of highway research, to form a portion of future basis for highway design?

I visited Vermont in November, 1927, a few days after the severe flood of that year, making—together with Mr. C. V. Jarvis and with Mr. G. H. Miller, District Engineer of the U. S. Bureau of Public Roads, a study of flood conditions. The direct highway damage in the small state of Vermont caused by three days flood, for the large part on non-navigable streams was approximately eight million dollars. I made a study of the indirect damage, through interruption to, and delay in business, due to highways not operative—and this indirect damage ran into several millions of additional loss.

I was in Southern California in the winter of 1915-16, as county engineer for San Bernardino County, California, when the excessive precipitation and runoff of that year occurred. The estimated highway damage, direct and indirect, caused by this four day's flood in Southern California, was approximately fifteen million dollars.

These are two typical instances, twelve years apart in time, and three thousand miles apart in geographic distance, the locations varying from the extreme northeast to the extreme southwest portion of the United States. Other instances are multiplied, occurring at intervals in nearly all parts of the country. The instances I refer to do not have to do with navigable streams throughout the United States. And they affect directly the cost of construction, maintenance and operation of public highways in the United States.

Mr. Eno touched on this problem of " precipitation and runoff " as one possibly not entirely to be included under the heading of " climate," but as a problem so closely allied and of such magnitude of importance as affecting highway construction, that he felt it should have further study.

I would point out, as Professor Eno has done, and I would like to place added emphasis, if it be possible, on the importance and on the practical value, of further study and cooperative constructive research, on these two general projects, (1) ground freezing, and (2) precipitation, runoff and stream control—both as affecting highway design and construction, in the United States.

Prof. D. P. Krynine, *Moscow Superior Technical School.* Professor F. H. Eno and the Committee on Structural Design of Roads are to be congratulated for having given start to a new branch of highway science, namely to the study of climatic conditions. Similar ideas have been formed in the Russian Highway Research Bureau, too; they may also be found in my Russian text book on Highway Engineering, edition 1929.

At the present time the science and practice of highway engineering have become international. A similar thing occurs for instance in North and South America, as I have learned from the brilliant talk of Mr. Pyke Johnson about the Pan American Conference of 1929. However, this circumstance obliges us to pay great attention to the difference of climatic conditions, and to the elaboration of new specifications accordingly. In the opposite case, a danger may arise either of producing an unsafe structure or of wasting a certain quantity of money in unnecessary work.

The effects of temperature, frost, sunshine, wind, humidity, precipitation, run-off and evaporation upon various phases of highway activities are interestingly discussed by Professor Eno. To this long list of climatic factors one more should be added namely, the influence of atmospheric pressure.

IMPORTANT DEVELOPMENTS WITH RESPECT TO SUBGRADE INVESTIGATIONS

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Senior Highway Engineer
AND
Henry Aaron
Junior Civil Engineer, U. S. Bureau of Public Roads

Among the important developments disclosed by subgrade investigations during the past year are the following:

1. A conception of the physical laws controlling the stability of subgrades.
2. A conception of the detrimental effects produced by elasticity in subgrades.
3. A conception of the physical laws controlling the occurrence of frost heave.
4. An understanding of the benefits which may be furnished by treating subgrades.
5. Suggested tentative grouping of subgrades based upon their performance.
6. The use of relatively few and comparatively simple laboratory tests which serve (a) to identify the dominating constituents of subgrade soils; (b) to disclose the important characteristics possessed by subgrade soils; and (c) to suggest the proper corrective measures to be used in pavement construction.

STABILITY OF SUBGRADES DEPENDS UPON BOTH INTERNAL FRICTION AND COHESION

(a) Internal friction or that portion of the resistance to shear which is dependent upon the external force applied, and

(b) Cohesion or that portion of the resistance to shear which is independent of the external force applied.

The significance of the term internal friction is illustrated by the performance of two pieces of sand paper when pressed together. Under these conditions they exert no resistance to being pulled apart. They exert resistance to sliding over each other, however, dependent upon the pressure with which they are pressed against each other.

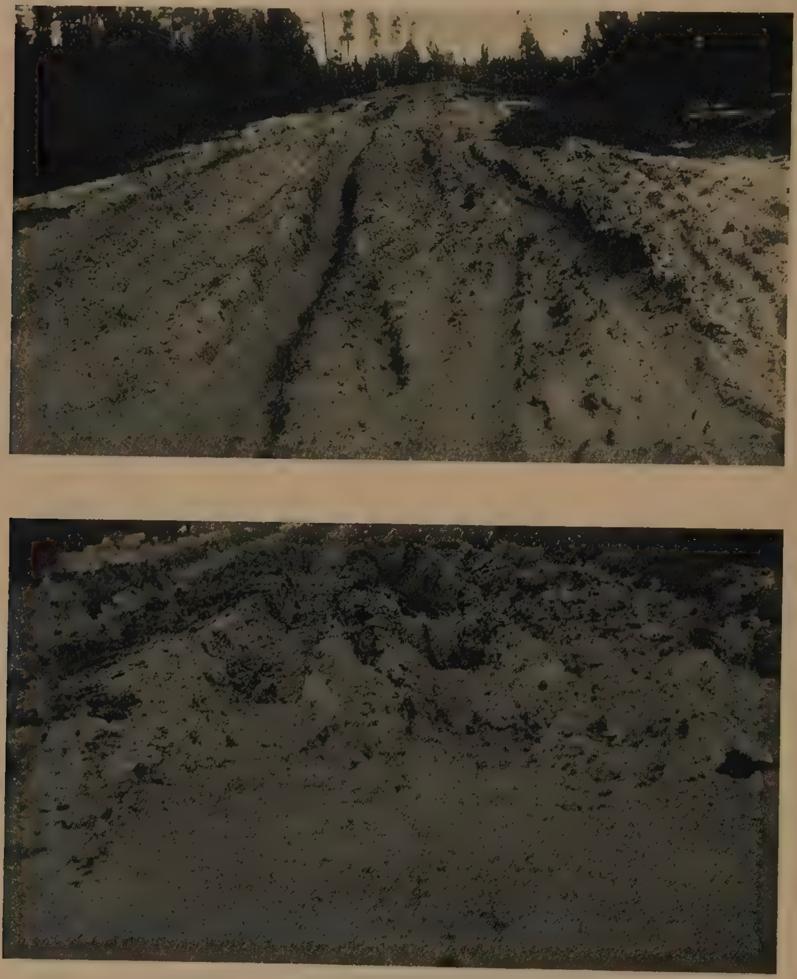
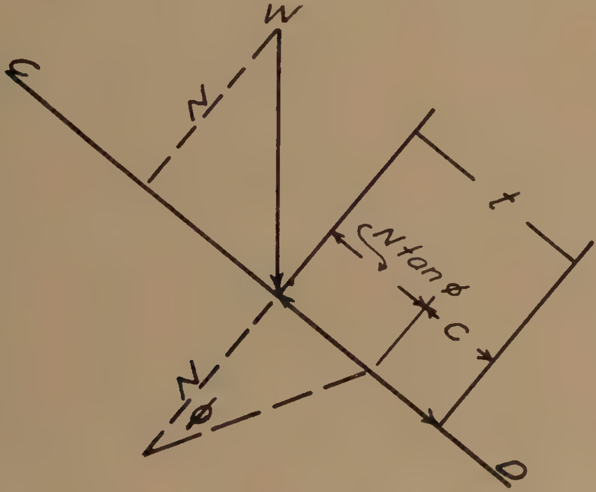


Figure 1. Unstable Road Surface Materials
Upper: Cohesionless sand
Lower: Soft cohesive clay

The significance of the term cohesion is illustrated by the performance of two pieces of fly paper when pressed together. Under these conditions they exert high resistance to being pulled apart due to stickiness or cohesion possessed by glue-like materials brought in contact with each other. The resistance which the pieces of fly paper exert to resist sliding over each other is furnished by surface stickiness instead of surface roughness as was furnished by the sand paper.

When a soil flows latterly, whether it pushes out from beneath a pavement edge or up into the interstices of a macadam, sliding occurs along either one or more planes existing in the soil. Furthermore this sliding occurs only because the force which produces sliding exceeds the shear resistance existing in the soil along the sliding plane or planes.



Assume for instance that the line *CD* represents any plane in the soil upon which a unit soil weight *w* is acting. A unit shear resistance *t* acting along the plane *CD* holds the soil (weight *w*) in equilibrium and prevents it from sliding.

The shear resistance *t* is composed of two parts: That furnished by the resistance of the soil grains to sliding over each other (sand paper effect) and that furnished by the cohesion (fly paper effect) existing between the soil particles. The resistance to sliding is dependent upon the angle of internal friction ϕ: that angle whose tangent equals the frictional resistance divided by the pressure normal to the sliding plane. The cohesion existing between soil particles is designated as cohesion in pounds per square feet, kg. per sq. cm., etc., existing between the soil particles.

When

$\phi =$ angle of internal friction.
$N =$ component of w, normal to CD.
$t = N \tan \phi + C$.

On this basis the Bureau of Public Roads, in its publication "Public Roads," vol. 10, No. 3, May, 1929,[1] * suggested a formula by means of which some conception could be obtained with respect to the influence exerted by both internal friction and cohesion upon stability.

TABLE I

VALUES OF c AND ϕ FOR DIFFERENT SOILS AND THEIR INFLUENCE UPON THE SUPPORTING VALUE, q FOR SEVERAL ASSUMED CONDITIONS
($b = 0.71$ FOOT AND $s = 100$ POUNDS PER SQUARE FOOT)

Soil	Cohesion, c, pounds per square foot	Angle of internal friction, ϕ Degrees	Supporting value, q,* pounds per square foot
Clay, almost liquid	100	0	400
Clay, very soft	200	2	864
Clay, soft	400	4	1,857
Clay, fairly stiff	1,000	6	4,982
Clay, stiff	1,500	8	8,050
Clay, very stiff	2,000	12	12,528
Silts, wet †	0	10	43
Sands, dry	0	34	768
Sand predominating with some clay	400	30	6,035
Sand-gravel mixtures, cemented ‡	1,000	34	17,838

* The values of q serve for demonstration purposes only and should not be used as a basis of pavement design.

† In silty soils, the angle of internal friction may vary between 10 and 30 degrees but the cohesion may be almost 0.

‡ In properly graded soils, depending upon the extent of their compaction, the angle of internal friction may exceed 34° and the cohesion may be considerably less than 1000.

According to Table I, the stability of clays may be increased enormously by reducing their moisture contents. Furthermore the stability of wet clays may be very appreciably increased by mixing sand with them and the stability of sands may be very greatly increased by mixing clay with them.

The supporting value of sand, for instance, possessing no cohesion and internal friction in amount represented by $\phi = 34°$, is 768 pounds per square foot. The supporting value of soft clay possessing cohesion in amount equal to 400 pounds per square foot and internal friction in amount represented by $\phi = 4°$, equals 1857 pounds per square foot. These two materials properly combined, however, are apt to furnish support exceeding in amount 6000 pounds.

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* Numbers refer to reports listed in the attached bibliography.

By use of the formula referred to it can be demonstrated also that with equal cohesion furnished by the clay, the stability of the subgrade increases with increase of internal friction furnished by the sand and for equal internal friction furnished by the sand the stability of the subgrade increases with increase of cohesion furnished by the clay.

Thus for instance when the cohesion equals 400 pounds per square foot the supporting value of the subgrade will equal either 2554 or 7596 pounds per square foot depending on whether the angle of internal friction equals 12° or 34°. And when the angle of internal friction equals 34° the supporting value of the subgrade will equal either 4182 or 17,838 pounds per square foot depending on whether the cohesion equals 200 or 1000 pounds per square foot.

According to available information sands possess a relatively high degree of internal friction. The greater the degree of angularity, sharpness or roughness possessed by the sand grains the greater is apt to be the degree of internal friction furnished by the sand. Silts possess internal friction in amounts varying between $\phi = 10°$ and $\phi = 30°$ depending upon the moisture content of the soil. Silts possess practically no cohesion. Clay and colloids furnish cohesion but possess internal friction in relatively small degree.

Practically this information suggests that:

1. Admixtures which serve to prevent the soil from taking up moisture, serve to prevent the soil from losing cohesion and therefore should serve to increase the stability of clays when used either as binders or subgrades.
2. Admixtures possessing cohesion should serve to furnish cohesion additional to that possessed by the soil. Admixtures of this type therefore should serve to increase (a) the stability of cohesionless silts when used either as binder or subgrades and (b) the stability of sands when used as subgrades.
3. Admixtures of granular materials possessing high resistance to sliding are apt to furnish stability in much greater amount than admixtures of granular materials possessing relatively small resistance to sliding.

ELASTICITY A DETRIMENTAL SUBGRADE CHARACTERISTIC

The ability to distort in a jelly like fashion (1) instead of permanently compacting when loaded, causes soils possessing elasticity to

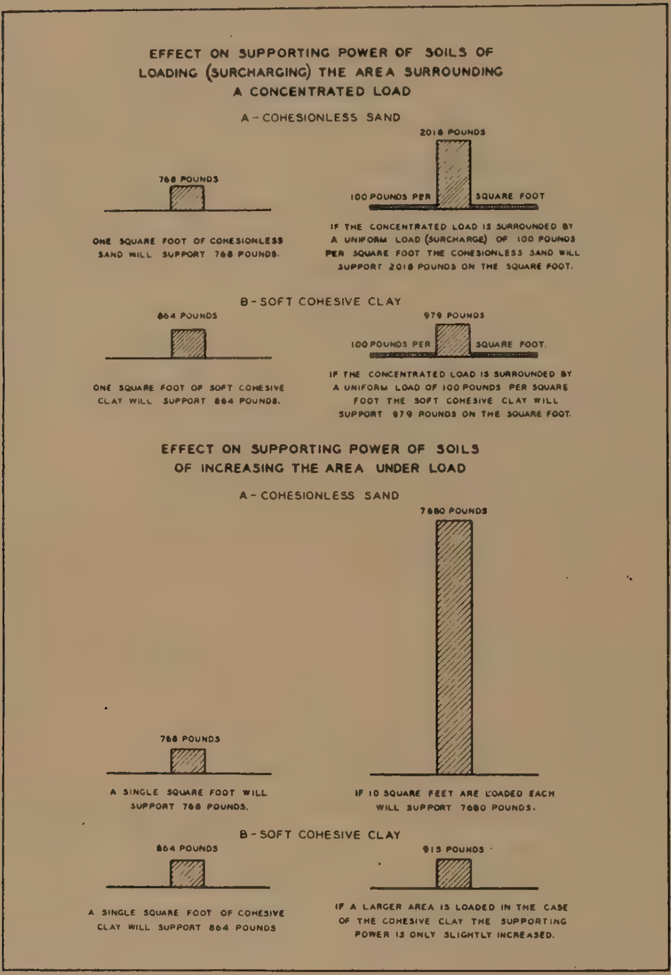

EFFECT ON SUPPORTING POWER OF SOILS OF LOADING (SURCHARGING) THE AREA SURROUNDING A CONCENTRATED LOAD
A - COHESIONLESS SAND
768 POUNDS
2018 POUNDS
100 POUNDS PER SQUARE FOOT
ONE SQUARE FOOT OF COHESIONLESS SAND WILL SUPPORT 768 POUNDS.
IF THE CONCENTRATED LOAD IS SURROUNDED BY A UNIFORM LOAD (SURCHARGE) OF 100 POUNDS PER SQUARE FOOT THE COHESIONLESS SAND WILL SUPPORT 2018 POUNDS ON THE SQUARE FOOT.
B - SOFT COHESIVE CLAY
864 POUNDS
979 POUNDS
100 POUNDS PER SQUARE FOOT.
ONE SQUARE FOOT OF SOFT COHESIVE CLAY WILL SUPPORT 864 POUNDS.
IF THE CONCENTRATED LOAD IS SURROUNDED BY A UNIFORM LOAD OF 100 POUNDS PER SQUARE FOOT THE SOFT COHESIVE CLAY WILL SUPPORT 979 POUNDS ON THE SQUARE FOOT.
EFFECT ON SUPPORTING POWER OF SOILS OF INCREASING THE AREA UNDER LOAD
A - COHESIONLESS SAND
7680 POUNDS
768 POUNDS
A SINGLE SQUARE FOOT WILL SUPPORT 768 POUNDS.
IF 10 SQUARE FEET ARE LOADED EACH WILL SUPPORT 7680 POUNDS.
B - SOFT COHESIVE CLAY
864 POUNDS
913 POUNDS
A SINGLE SQUARE FOOT OF COHESIVE CLAY WILL SUPPORT 864 POUNDS
IF A LARGER AREA IS LOADED IN THE CASE OF THE COHESIVE CLAY THE SUPPORTING POWER IS ONLY SLIGHTLY INCREASED.

Figure 2. Sketch Illustrating How Cohesionless Differ from Cohesive Soils When Used as Subgrades

Upper: The appreciable increase in the unit support that is furnished by cohesionless soils when surcharged adjacent to the loaded area is not furnished by soft cohesive soils under the same conditions.

Lower: The appreciable increase in unit support that is furnished by cohesionless soils when the loaded area is increased in width is not furnished by soft cohesive soils under the same conditions. (Public Roads, Vol. 10, No. 3, May, 1929.) Also, the safe slope of cohesionless soils is constant irrespective of both the height of fills and depth of cuts. The safe slope of cohesive soils in contrast decreases as either the height of fills or the depth of cuts increases. (Public Roads, Vol. 10, No. 10, December, 1929.)

rebound upon the removal of load. This performance is caused by the presence of soil constituents such as mica and organic matter which possess abnormally high porosity when dry. Furthermore, the pores of these materials when wet are apt to contain air in appreciable amounts and this also is productive of elasticity. Clay soils flocculated due to the presence of certain chemicals also are apt to possess elastic properties under certain conditions.

Cohesionless soils possessing elasticity are apt to prove troublesome when both dry and wet. Cohesive soils of this group, in contrast, may exhibit elastic properties only when containing certain amounts of moisture.

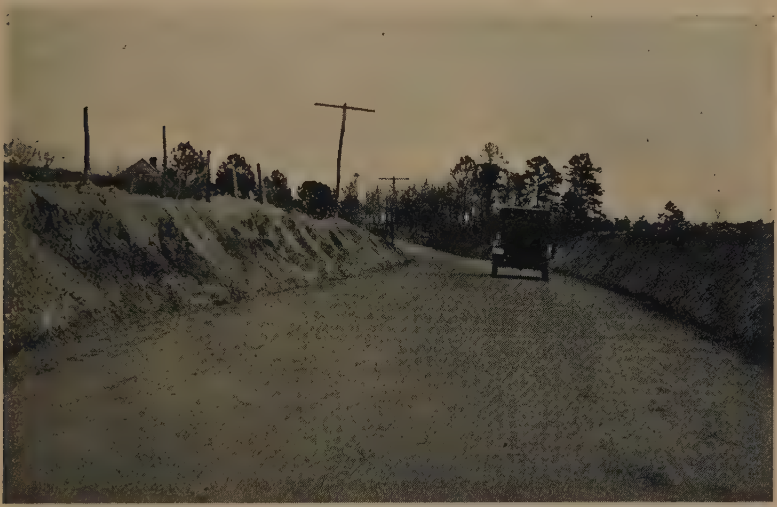
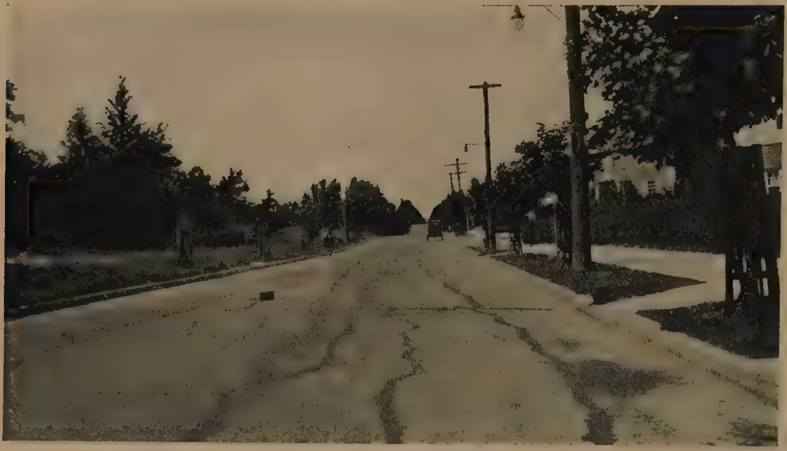


Figure 3. Highly Stable Mixture of Sand, Silt and Clay Possessing Both Internal Friction and Cohesion in Proper Amounts

A knowledge of the existence of elasticity in subgrades prior to pavement construction might be the means of saving the cost of either a concrete or a macadam pavement.

After thorough rolling according to the current procedure for instance, a subgrade of the elastic type if it possesses cohesion is apt to retain a certain degree of compaction. A slight wetting under these conditions such as is furnished by freshly deposited concrete may cause a non-uniform rebound of the subgrade. This combined with water loss from the concrete due to absorption by the soil may cause pavements to crack excessively during the setting period of the concrete.



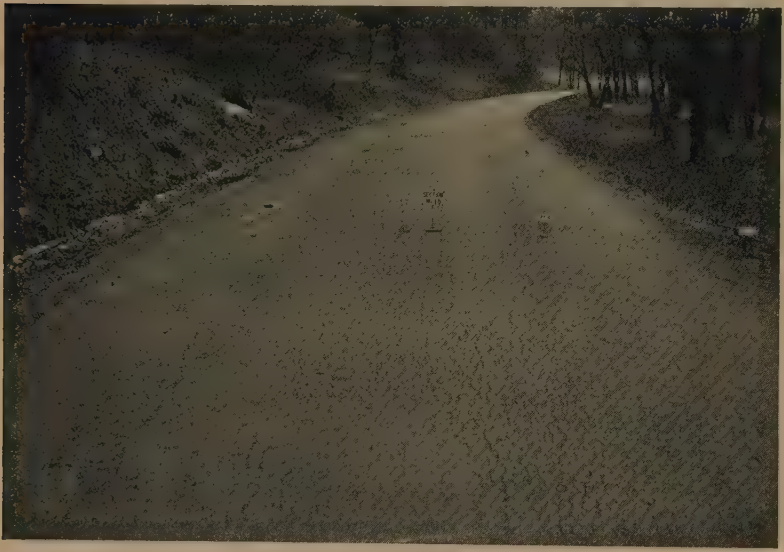


Figure 4. Detrimental Effects of Elasticity in the Subgrade

Upper: Cracking apt to occur in concrete pavement during setting period. Note how cracks occurred apparently over ruts formed in the subgrade by the concrete mixer.

Lower: Alligator hide cracking apt to occur in macadam laid on elastic subgrade. After cracking of this type occurs, water enters the subgrade, softens it, and permits the subgrade soil to work into the interstices of the macadam. In this instance the subgrade soil has penetrated the road surface to within about an inch of the top. (Photograph furnished by I. B. Mullis.)

Likewise movements of both heavy material trucks and mixing apparatus adjacent to pavements laid on elastic cohesionless subgrades may cause distortions of the soil supporting the freshly laid concrete productive of pavement cracking. Cracking of this character might remain in microscopic form for an appreciable period of time. Except during the setting period of the concrete, elastic subgrades may not be detrimental to concrete pavements.

The presence of elasticity in subgrades may prevent macadam pavements from acquiring adequate bond during construction and from retaining it subsequently. Under these conditions macadams are apt to develop "alligator hide" cracking through which water may pass and cause the subgrade soil to soften and to penetrate the voids of the macadam thus causing the macadam to fail.

Practically this information suggests the following with respect to the preparation of elastic subgrades:

(a) Stabilize elastic subgrades by treatment as for instance stage construction before attempting to construct a macadam pavement.

(b) When absence of cohesion in the subgrade (highly micaceous sands, etc.) interferes with the construction of pavements, admixtures of cohesive materials (clays) may prove beneficial.

(c) Make the subgrade both as uniformly dense and as uniformly moist as possible by breaking up all clods with a light weight roller (about 3 tons) harrowing, and possible sprinkling.

(d) Attempt to prevent excessive moisture change in the soil during the setting period of the concrete by either placing tar paper or similar material or applying bituminous coatings on the subgrade.

FROST HEAVE APT TO BE DETRIMENTAL IN PERMEABLE SILTS

Frost heave which is due to the formation of well defined ice layers [2] instead of to the expansion during freezing of the moisture originally contained in the frozen soil layer is due to three physical phenomena:

(a) The ability of water particles contained in soil pores larger than about capillary dimension to freeze at either normal freezing or slightly less than normal freezing temperatures (— 1° — 4° C).[3]

(b) The ability of water particles contained in soil pores of capillary dimension to resist freezing at abnormally low temperatures (as low as — 70° C).[3,4,5]

(c) The ability of water particles of freezable size, during the process of freezing to draw to themselves from adjacent fine capillaries the small particles of water which individually do not freeze at ordinary freezing temperatures.[3,4,5] When drawn to the existing ice crystal, however, these small water particles freeze and increase the size of the original ice crystal. Continuation of this process causes the original ice crystals to increase in size as long as they are being furnished small water particles drawn up through the fine capillaries from the ground water supply.

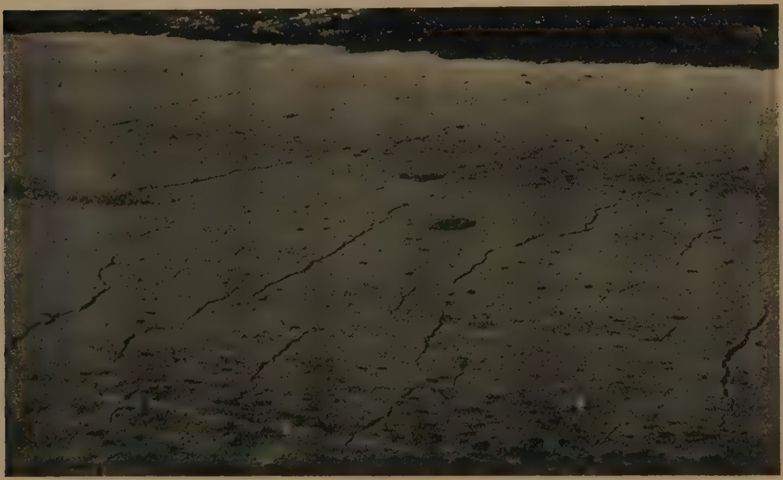


Figure 5. Early Cracking in Concrete Pavement Eliminated by Placing Tar Paper on Subgrade. (Photograph furnished by R. W. Crum.)

In sands no important frost heave occurs[2] because practically all of the contained water freezes at normal freezing temperatures[5] and small unfrozen water particles do not exist in amount sufficient to cause the frozen particles to suffer appreciable growth.

Permeable silts which are capable of raising water rapidly and through distances of considerable amount are apt to suffer very important frost heave. These silts may be capable of raising by capillarity at the rate of approximately 0.9 pound of water per square foot of soil per day when the ground water elevation is 4 feet below the surface of the ground.[6] At this rate a layer of water approximately 5½ feet in depth would be raised during one year.

The capillary raise may be as high in cohesive clays as in silts. The speed with which water rises in clays, however, is much less than in silts. Consequently, in dense clay soils with low ground water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals is apt to dry out and shrink due to the loss of moisture.[7] The ground water elevation in clays, therefore, must be comparatively high in order that important frost heave may occur. Or the clay must be wet due to water absorption, with manipulation on top of the subgrade.

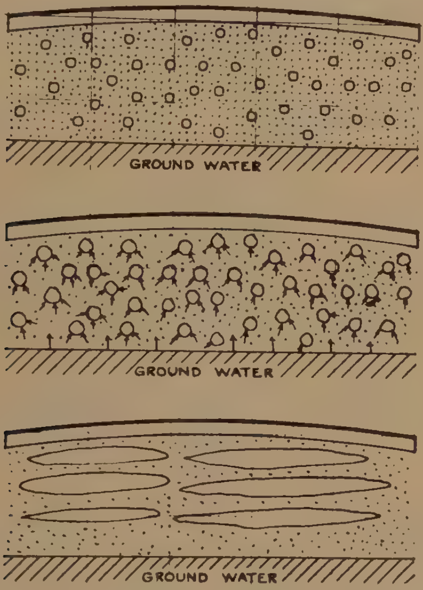


Figure 6. Diagrammatic Sketch of Frost Heave

Upper: Circles represent water particles of size freezable at normal freezing temperatures. Dots represent water particles which fail to freeze at normal freezing temperatures.

Middle: Shows frozen water particles (circles) growing by drawing to themselves the small unfrozen particles (dots) and the flow of capillary moisture from the ground water to fill the soil pores vacated by the small unfrozen water particles.

Lower: Resulting layers of ice which cause the road surface to heave.

Additive to the detrimental effects of heaving is the softening of the subgrade during thaws due to the liberation of excessive amounts of water trapped by both frozen under-soil and frozen shoulders.

Due to a variety of causes frost depth under roadways is apt to be greater than under shoulders. Also thawing is apt to occur first under the roadway; therefore, water liberated in this manner cannot penetrate the frozen soil along the edge of the roadway and reach the drain located there. Thus the drains must be placed at the location of maximum frost depth which is likely to coincide with the location of initial thawing. In this respect the center drain according to L. L. Allen, Minnesota State Highway Department, serves (a) to lower the moisture content of the soil before freezing and (b) to provide for the disposal of water liberated by thaws.

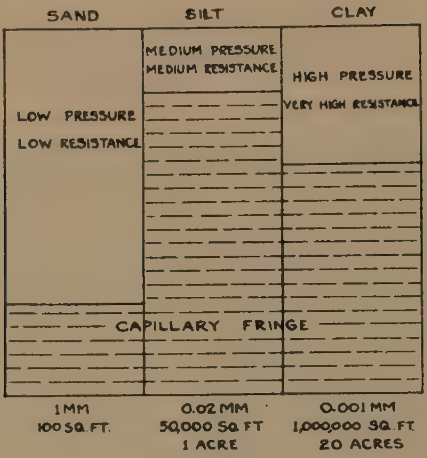


Figure 7

Sketch showing why drains must be placed very deep in order to prevent frost heave in silts. The capillary rise which feeds the growing frost crystal depends upon (a) the capillary pressure which forces water up into the soil pores, and (b) the frictional resistance offered by the soil pores to the flow of water. The capillary pressure increases with decreasing soil particle size. The frictional resistance to flow increases with increase in surface area of soil particle. Silts with grain size of about 0.02 mm. seem to possess the proper combination of both capillary pressure and frictional resistance productive of high capillary rise. In silts and mucks (see Figure 13) drains must be placed at a comparatively great depth. The areas shown at the bottom of the sketch represent the total surface area of soil particles contained in one cubic foot of soil mass.

Also according to both A. C. White [8] and J. T. Henton [9] drains backfilled with gravel and placed longitudinally under the center of gravel road surfaces are effective for curing frost boils in both Minnesota and Wisconsin.

Especially important with respect to frost heave (according to W. I. Watkins, U. S. Bureau of Chemistry and Soils, E. A. Willis, U. S. Bureau of Public Roads, and John Morton, New Hampshire State Highway Department) are soils consisting of sand to a depth of as much as 12 or 18 inches underlain with silt zones which sag and permit water to accumulate.

Practically this information suggests that:

1. Any admixture which serves to reduce the moisture content possessed by the subgrade soil is apt to reduce the extent of frost heave.
2. Any treatment which serves to prevent water entering the subgrade from above may serve at the same time to prevent increase in the moisture content of the subgrade to depths depending upon the permeability of the subgrade soil. Consequently a treatment of this character may serve to reduce the extent of frost heave suffered by the subgrade due to water absorption from above.

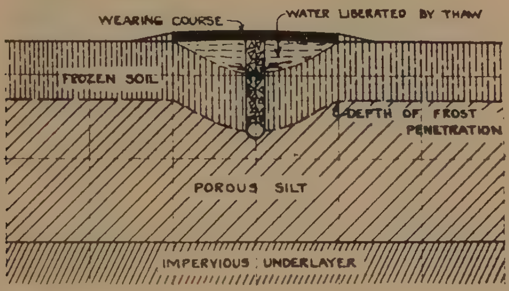


Figure 8. Sketch Showing How the Center Trench Serves to Dispose of the Water Liberated by Thaws

3. Any admixture which serves to furnish cohesion in silts is apt to reduce also the extent of frost heave suffered by subgrades consisting of silt.
4. Drains backfilled with porous material placed longitudinally under the center of gravel road surfaces, and in depth slightly greater than the depth of frost penetration should serve to (a) reduce the extent of frost heave in silts by reducing the moisture content possessed by the soil before freezing and (b) prevent loss of stability in silts by disposing of water liberated during thaws.

THE FUNCTION OF BITUMINOUS SUBGRADE TREATMENTS WITH RESPECT TO THE CONSTRUCTION OF MACADAM PAVEMENTS ILLUSTRATES BENEFITS APT TO BE FURNISHED BY SUBGRADE TREATMENTS

Primarily the failure of macadams on other than elastic subgrades is due to the subgrade soil penetrating the interstices of the macadams. In order that this may occur, (a) the subgrade soil must

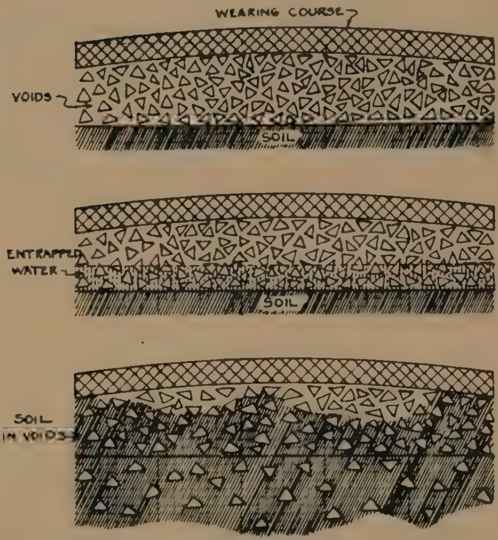

WEARING COURSE
VOIDS
SOIL
ENTRAPPED WATER
SOIL
SOIL IN VOIDS

Figure 9. Sketch Illustrating the Failure of Macadam and Porous Base Courses

Upper: New roads with pores in base course.
Middle: Water trapped in pores of base course.
Lower: Penetration of subgrade soil into pores of base course.

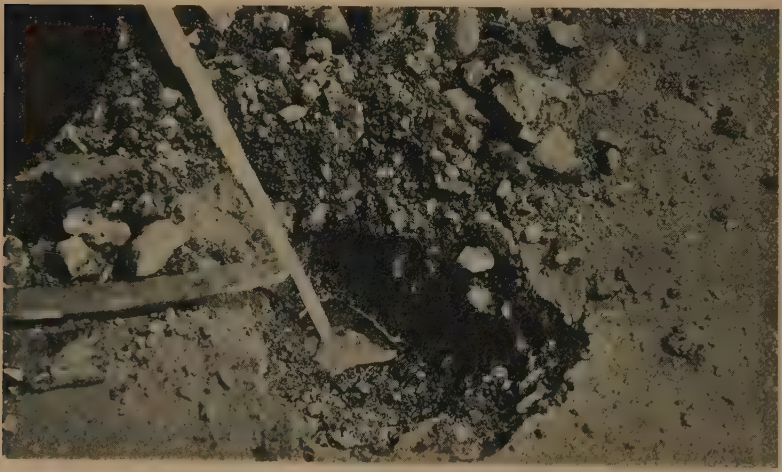
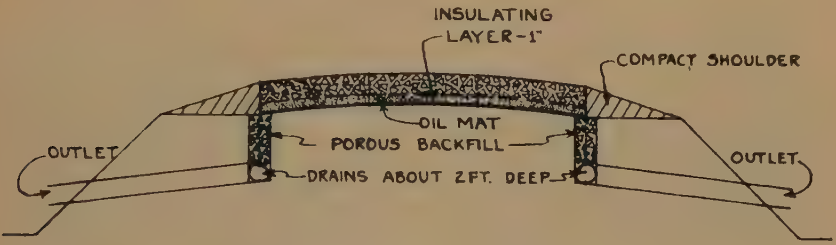


Figure 10. Entrapped Water Which Entered a Hole Dug in a Macadam Base in 3 Minutes. The Pavement Rested on a Fill 10 Feet High and the Hole Was Dug 7 Days After a Rain Storm

be wet and (b) voids in appreciable amount must exist in the under course of the macadam.

Applying bituminous material on the subgrade and covering with a cushion of granular material before constructing the macadam, supplemented by drainage as specified for stage construction, is suggested to prevent failure of this character.

Sands are not likely to work up into the interstices of macadams. The object of treating them in this case therefore is to furnish the cohesion necessary for stabilizing cohesionless materials. The increased stability furnished by treatments of bituminous materials which penetrate the sands to a depth of $1\frac{1}{2}$ to 2 inches may serve to permit a reduction of appreciable amount in the thickness of the macadam wearing course.



DRAINAGE IN SAGS

Figure 11. Preventive for Macadam Failures. Treat Subgrades with Bituminous Material and Cover with Granular Material. Drain All Sags in the Grade

The object of the treatment on clays is to furnish impermeable top coating. Therefore in this case the depth of soil penetrated should be such as to furnish adequate bond between the bituminous coating and the dense subgrade soil. It is possible that bituminous materials which penetrate the subgrade soil to a depth of 5/16 to $\frac{3}{8}$ of an inch will furnish the bond in required amount.

Why these skin treatments on clay subgrades may furnish benefit equal to that furnished by foundations of appreciable thickness is apparent. Clay soil, until after it flows laterally, cannot penetrate the interstices of the macadam. Clay soil cannot flow laterally until after it has attained very low cohesion due to water absorption. Clay soil in the plastic state cannot absorb water when present unless the clay is manipulated. Until the upper skin of clay in contact with the under stones of the macadam, is manipulated in the presence of water and softened, the soil located beneath this upper skin is not enabled to absorb water and soften. The water which is apt to cause this upper skin of an impervious clay soil to soften must of necessity be water resting on the surface of the subgrade and trapped in the interstices of the macadam. The bituminous treatment prevents this water from both coming in contact with and softening the upper skin of the subgrade soil. The cushion layer of granular material serves to fill the voids in the macadam.

Consequently, the subgrade treatment serves

1. To eliminate the voids into which the clay would enter.
2. To eliminate the voids in which water would collect on top of the subgrade.
3. To prevent water resting on top of the subgrade from both entering and softening the subgrade soil.

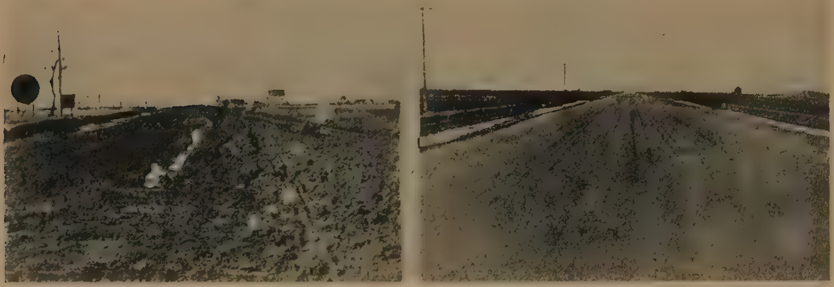


Figure 12. Effect of Bituminous Treatment Covered with Gravel Blotter Layer on Gumbo Soil

Left: Gravel road on gumbo after spring thaw.
Right: Thin gravel course on bituminous treated gumbo soil after spring thaw.
(Photographs furnished by F. C. Lang.)

SUBGRADES TENTATIVELY ARRANGED IN GROUPS

With respect to their performance the various subgrades may be tentatively arranged in groups[1] as follows:

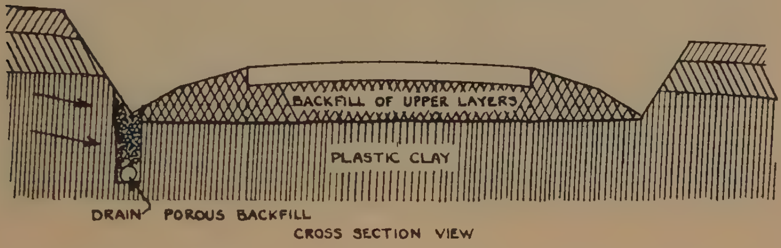
Uniform Subgrade

Group A-1. Well graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface treated or when used as a base for relatively thin wearing courses. Represented by the excellent top soils of Georgia.

Group A-2. Coarse and fine materials, inferior binder. Highly stable when fairly dry. Apt to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer. Represented by poorly graded top soils.

Group A-3. Coarse material only, no binder. Lacks stability under wheel loads but unaffected by moisture conditions. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements. Represented by the Florida sands.

Group A-4. Silt soils without coarse material, and with no appreciable amount of clay. Apt to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. Apt to cause cracking in rigid pavements due to frost heaving and failure in flexible pavements due to low support. Represented by New Hampshire silts.



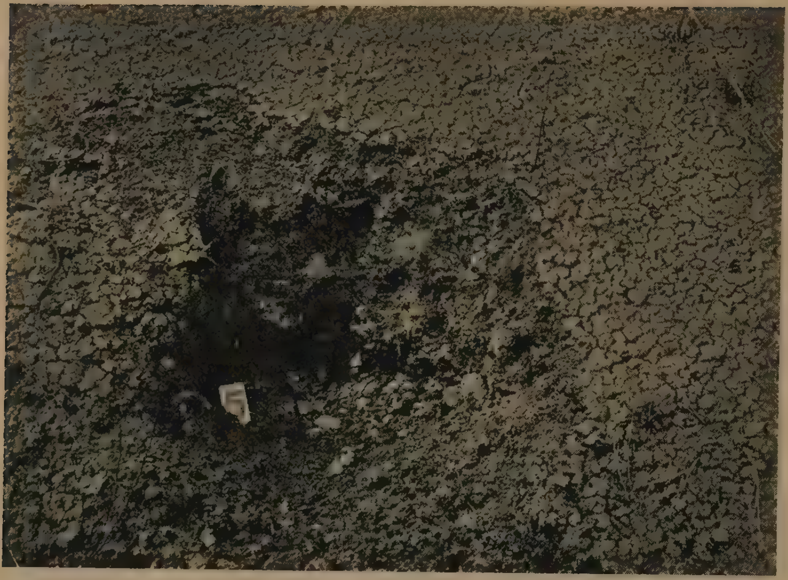
SUBGRADE TREATMENT FOR PAVEMENT ON VERY DENSE PLASTIC CLAY WHICH HAS CAUSED CONSIDERABLE DISTORTION IN PAVEMENTS

Figure 13

Sketch showing how soils possessing shrinkage properties in high degree should be treated in cuts: Excavate to an appreciable depth below grade; (b) backfill with good top soil material; (c) use underdrains instead of ditches; (d) coarse materials such as gravel or rock should not be placed directly on soils of this character. Soils of this character are not apt to prove satisfactory when used in fills of appreciable height.

Group A-5. Similar to Group A-4, but furnishes highly elastic riding surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards. Represented by highly micaceous soils of North Carolina.

Group A-6. Clay soils without coarse material. In stiff or soft plastic state absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams.



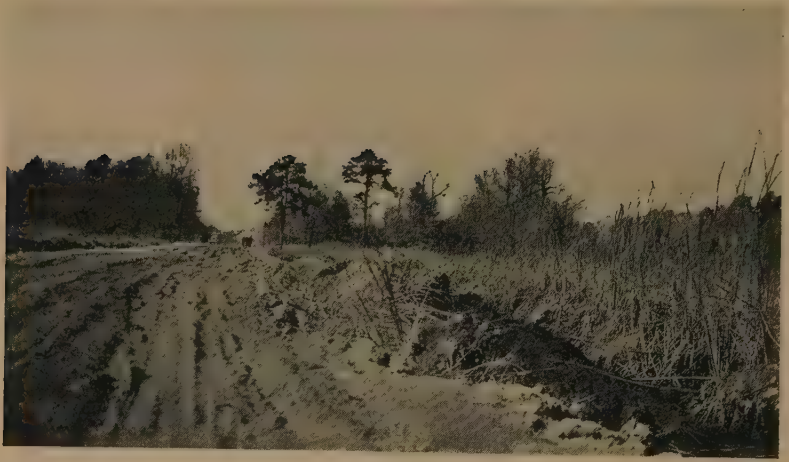


Figure 14

Upper: Typical appearance of Group A-6 or A-7 soils possessing shrinking properties in high degree.

Lower: Fill settling through a Group A-8 subgrade. Mound to right of roadway shows muck displaced by fill material.

(Photograph furnished by J. D. Fauntleroy.)

Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are apt to cause cracking in rigid pavements. Represented by the Mississippi gumbo.

Group A-7. Similar to Group A-6, but when moist, deforms quickly under load and rebounds appreciably upon removal of load. Thus, lacks firmness in support, similar to subgrades of Group A-5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A-6 subgrades. May cause concrete pavements to crack before setting. Represented by Illinois bumbo and typical adobes.

Group A-8. Very soft peat and muck incapable of supporting a road surface without being previously compacted or displaced by a fill. Represented by Minnesota peat bogs.

Non-Uniform Subgrades

Soils of this group cause concrete pavements to crack or fault excessively and flexible type surfaces to fail or develop rough riding surfaces.

Group B-1. Non-uniform natural ground due to abrupt variation in soil characteristics, or soil profile, or to frequent change in field conditions.

Group B-2. Non-uniform subgrade due to non-uniform composition of fill.

Group B-3. Non-uniform subgrade consisting in part of natural ground and part of fill materials.

SUBGRADE SOIL CONSTANTS INDENTIFY IMPORTANT SUBGRADE CHARACTERISTICS

The mechanical analysis which furnishes information only with respect to soil particle size within rather wide limits, is inadequate to furnish complete identification of subgrade characteristics. That fraction indicated as sand for instance by the mechanical analysis may be either a round grain sand which flows under low moisture content (mere lubrication of grain surface) or an angular grained sand which flows only due to hydrostatic uplift. Likewise, that fraction indicated as silt by the mechanical analysis may be compressible inert material or a micaceous material possessing elasticity in detrimental amounts. Also those fractions designated as clay may differ

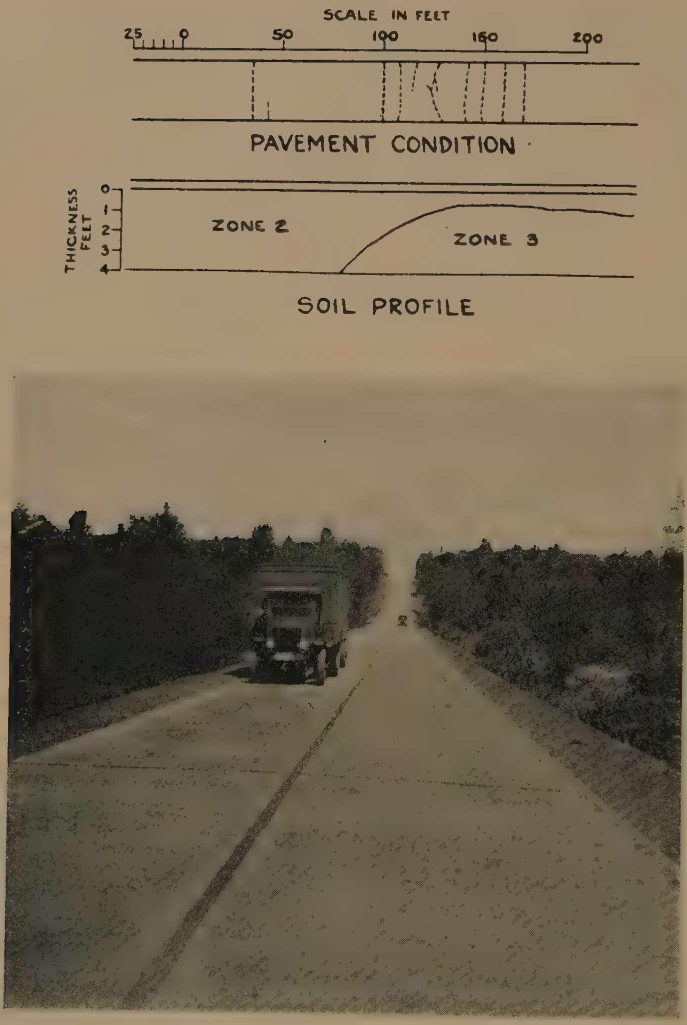

SCALE IN FEET
25
0
50
100
150
200
PAVEMENT CONDITION
THICKNESS FEET
0
1
2
3
4
ZONE 2
ZONE 3
SOIL PROFILE

Figure 15

Upper: Cracking in concrete pavement due to non-uniform B-1 subgrade. Zone 3 is an impervious compact soil. Similar cracking would be apt to occur if Zone 3 had been rock or ledge.

Lower: Concrete pavement divided by means of crack control into slabs 10 feet wide by 20 feet long. Crack control of this type is apt to prevent detrimental effects of cracking due to non-uniform subgrade support.

widely with respect to performance. Therefore, in order to identify the characteristics of subgrade soils, one must employ constants which disclose the degree to which soils possess particular physical properties. The constants suggested for accomplishing this purpose are the lower liquid limit, the lower plastic limit, the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, the field moisture equivalent, the shrinkage ratio, the volumetric change, the lineal shrinkage and the slaking value.

These constants may be briefly defined as follows:

Lower Liquid Limit. Minimum moisture content of the soil in percentage of the weight of the dry soil at which a standard number of shocks of standard intensity just begins to transform the lower limit of two sections of the soil cake into the liquid state.

Lower Plastic Limit. The minimum moisture content in percentage of weight of the dry soil, at which the soil can still be rolled out into threads 3/16 inch in diameter without the threads breaking into pieces.

Plasticity Index. The difference between the lower liquid and the lower plastic limits.

*Shrinkage Limit.** Mathematical computation according to the formula

$$S = w - \frac{V - V_o}{W_o} \text{ in which}$$

S = shrinkage limit, moisture content in percentage of weight of dry soil.
w = moisture content of the thoroughly wet soil cake in percentage of weight of dry soil.
V = volume of the wet soil in cubic centimeters when moisture content equals w.
V_o = volume of the thoroughly dried soil cake in cubic centimeters.
W_o = weight of the thoroughly dried soil cake in grams.

Centrifuge Moisture Equivalent. Moisture content in percentage of weight of the dry soil retained by a soil sample when subjected to a centrifugal force of 1000 times gravity after being allowed in the dry state to soak in water under zero pressure for a period of six hours.

Waterlogging in the centrifuge moisture equivalent consists of free water being retained on top of the soil cake after centrifuging.

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* See author's discussion of Professor Krynine's report, these proceedings.

Field Moisture Equivalent. Moisture content in percentage of weight of the dry soil at which a drop of water is not absorbed by the smoothed surface of the sample when its moisture content is being gradually increased by slowly adding water to the sample beginning with soil in the air-dried state.

*Shrinkage Ratio.** The weight of the thoroughly dried soil cake in grams (W_o) divided by the volume of the thoroughly dried soil cake in cubic centimeters (V_o).

Volumetric Change. Decrease in volume in percentage of the dried soil cake suffered by a soil sample when due to evaporation its moisture content is reduced to zero from an amount equal to some arbitrary moisture content.†

Lineal Shrinkage. Decrease in length in per cent of length of the wet soil cake suffered by a soil sample when due to evaporation its moisture content is reduced to zero from an amount equal to the field moisture equivalent.

Slaking Value. Time in minutes required for a soil sample after being wetted, molded into a cake, compressed and air dried to disintegrate when immersed in water.

The laboratory tests which furnish the subgrade soil constants are the centrifuge moisture equivalent, the shrinkage limit, the lower liquid limit, the lower plastic limit, the slaking value and the field moisture equivalent.

A combined sieve and Bouyoucos hydrometer method is used for determining the mechanical analysis.

Both the testing of subgrade soils and the interpretation of soil test results requires the services of one specially trained in these matters and will not be discussed in this résumé.

CONCLUSIONS PRESENTED

Based upon the results furnished by the subgrade investigations carried on by the U. S. Bureau of Public Roads and cooperating agencies the following conclusions have been presented:[1]

Subgrade Support

1. The support furnished by different subgrade soils varies widely with regard to intensity of support (high or low), character

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*Unit volume reduction per unit moisture content reduction for moisture contents above the shrinkage limit.

†A moisture content equal to the field moisture equivalent is used when the lineal shrinkage is to be estimated from the volumetric change.

of support (firm or elastic), and uniformity of support (constant or variable intensity of support), and thus permits a differential of considerable amount in pavement requirements.

2. The character of support afforded by subgrade soils is dependent upon soil constituents such as sand, clay, silt, mica, organic matter and diatoms, and upon the structure of the soil and the field conditions under which it exists.
3. The intensity of support in uniform subgrades depends upon the cohesion and internal friction in the soil, the area of load distribution, and the weight of superimposed pavement.
4. High subgrade support requires either high cohesion or high internal friction, preferably both combined. High cohesion is characteristic of clays with stiff consistency; high internal friction, of well graded sands; and a combination of both internal friction and cohesion of well-graded sands with a binder.
5. The greater the internal friction the more will the support of the subgrade be improved by distributing the load over a larger area or by increasing the dead weight of the pavement.
6. Increasing the intensity of subgrade support permits a reduction in the thickness of nonrigid foundations and increases the factor of safety against " breakage " * but not necessarily against longitudinal and transverse cracking in rigid pavements.
7. Eliminating the elastic rebound in subgrades permits the construction of nonrigid pavement types otherwise not suitable.
8. Increasing the uniformity of subgrade support reduces the extent of transverse and longitudinal cracking in rigid pavements and may provide against failure in nonrigid types.

Drainage

9. Side ditches or deep trenches may intercept water furnished by porous strata and may serve to lower the ground-water level. Thus they may be effective for preventing failure in side-hill fills, for increasing the subgrade support, and for reducing the extent of frost heave.
10. Tile laid in trenches filled with porous material and placed under the edges of pavements serve to intercept water entering from the sides and to prevent the accumulation on top of the subgrade. Thus, they may serve the same purposes as deep

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* Cracking of the pavement into small pieces due to load and uniform support.

side trenches and, in addition, reduce the extent of softening of the subgrade due to saturation from the top. Also they provide against the separation of cracked rigid slabs, due to successive freezing of water trapped by impervious shoulders.

Subgrade Surface Treatments

11. Subgrade surface treatments consisting of oiling and blanket courses serve to prevent the infiltration of soft subgrades into superimposed pavements and thus they may very appreciably increase the service value of nonrigid pavements laid on fine silt or clay subgrades.

Subgrade Treatments

12. Subgrade treatments serve primarily to increase the intensity of subgrade support, although, depending upon conditions, they may also increase the uniformity of subgrade support. Thus, they are beneficial primarily only with respect to the construction of nonrigid pavements.
13. Intensity of subgrade support is increased by adding cohesive materials to sands, granular materials to clays, and both cohesive and granular materials to silts. Adding moisture capacity reducers to retain high cohesion, and granular materials to supply internal friction, to clays may increase the benefits furnished by the addition of granular materials alone.
14. Subgrade treatments should be planned on the basis of the information furnished by laboratory tests concerning the soil properties.

Base Courses

15. Compacted base courses serve primarily to furnish load distribution in nonrigid pavements.
16. Except when porus base courses extend from the bottom of the pavement to the top of a rock subgrade, compacted base courses may be the more efficient.
17. The penetration of the underlying clay into the voids of porous base courses may furnish conditions of support which, due to their lack of uniformity, may be extremely undesirable.
18. Base courses may reduce the extent of longitudinal cracking which would otherwise occur in rigid pavements. This ceases to be a benefit in pavements which contain a center joint or groove.

19. Except when lack of uniformity in subgrade support is caused by conditions which exist in close proximity to the pavement, the efficiency of base courses for decreasing the extent of primary and secondary transverse cracking in rigid pavements is questionable.

Pavement Design

20. " Beam " strength is desirable in pavements laid on soils whose low support is due to a lack of internal friction.
21. " Beam " strength is not a necessity in pavements laid on soil whose low support is due to the absence of cohesion.
22. " Beam " strength is not an essential requirement for wearing courses on Group A-1 subgrades.
23. Increasing the thickness of rigid pavement furnishes an increase in the factor of safety against the occurrence of " breakage " but not to an appreciable extent against the occurrence of transverse cracking due to lack of uniformity in support.
24. Crack control and steel reinforcement serve to eliminate the undesirable effects caused by such cracking as cannot be prevented in rigid pavements. Their use permits a predetermination of the ultimate size of slabs, free from additional visible cracks, and also prevents faulting and separation of the slabs.
25. A subgrade survey record, to be of practical use in the design of highways, should consist of a map showing the ground-water level and the soil profile together with a description of the physical properties (both laboratory and field) of the various soils which comprise the profile.

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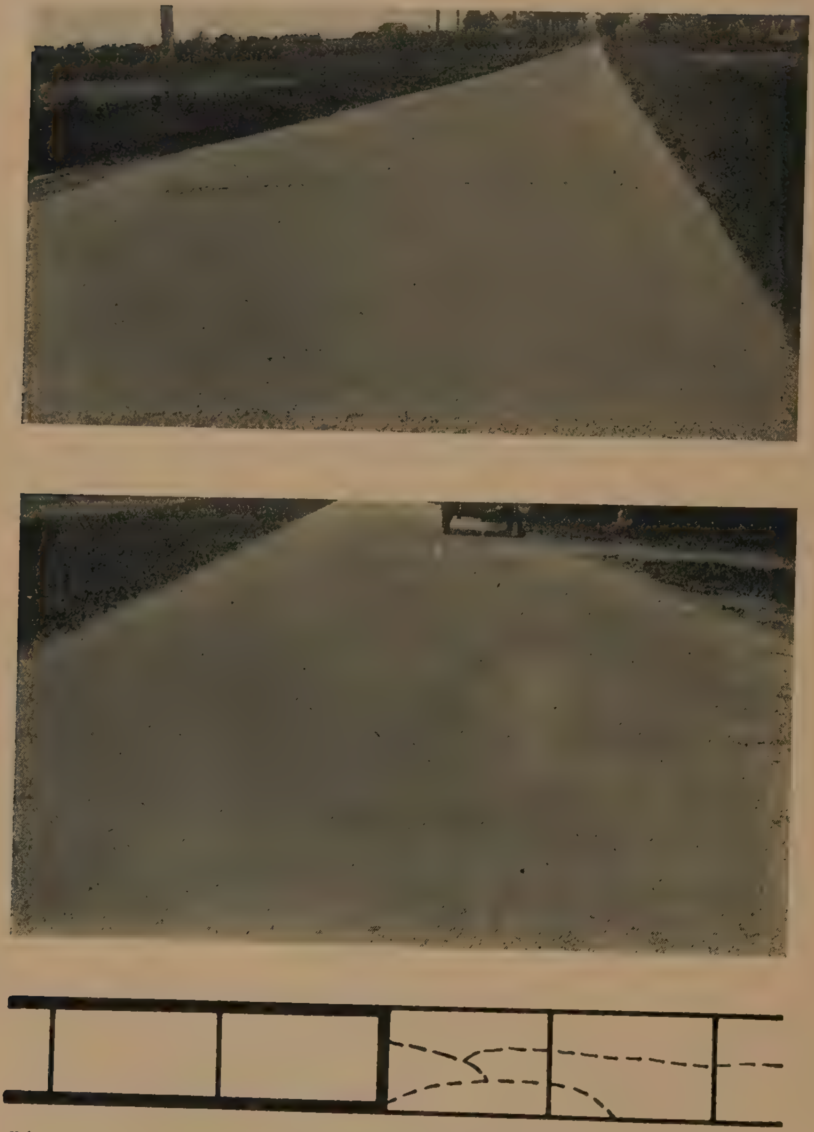

Figure 16. Effect of Light Weight Reinforcement for Reducing the Extent of Cracking in Pavements

Upper: Reinforced concrete slabs without cracks.
Lower: Badly cracked adjoining plain concrete slab.
Sketch at bottom: Map of pavement slabs. Those with heavy border are reinforced and contain no cracks. Broken lines show cracks in adjoining plain concrete slabs.

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DISCUSSION

ON

IMPORTANT DEVELOPMENTS WITH RESPECT TO SUBGRADES

Mr. J. A. Sourwine, *U. S. Bureau of Public Roads.* I want very briefly to pay tribute to Mr. Hogentogler and Mr. Terzaghi, who in their outline of new soil classifications, practically adapted for use in highway design, have achieved one of the boldest and one of the most basically valuable accomplishments which have come to highway research over a period of many years. Their thoroughly practical soil groupings for use as a basis in highway design make possible further detailed study which under the old forms of soil grouping and classification, developed primarily for other than highway purposes, was not possible. By their new practical groupings they make possible the application of much knowledge which we now possess, but which has previously been very difficult of practical application for highway use. The possibilities of further cooperative, constructive research along the general lines as outlined by Mr. Hogentogler, are tremendous. His present study, in my judgment, marks an "epoch" in the development of soil research, as applied to public highway design and construction.

CONTROL OF MATERIALS AND MIXTURES FOR CONCRETE FOR PAVEMENTS

By R. W. Crum, *Director*
Highway Research Board

The control of the various operations involved in producing concrete of which each batch is like every other batch within narrow limits seems to be complicated and difficult, but when the various necessary steps are taken in order, it is found that none of them need be very intricate, and that excellent results can be secured.

This paper is descriptive of the methods that are being used by the Iowa Highway Commission to produce concrete for pavements that is high in strength, uniform in quality, and of definite, predetermined composition.

The methods used or similar ones can be used anywhere, if in the hands of competent inspectors. Adequate control of these important technical operations can be best secured when the inspectors are qualified by their training to understand the reasons for the various operations.

The observations made and data secured during the construction of 700 miles of concrete pavement in 1928 demonstrate the benefits derived from such careful control and point out features in which further refinement and information are needed.

The Iowa specifications for mixtures and the methods used for controlling the operations of the making of the concrete are based upon two fundamental considerations. These are quality and definiteness of requirements as to quantities of materials. Contracts for the pavement jobs including all materials are awarded at public letting to the lowest responsible bidder. An element of very great importance in bidding is the estimate of the quantities of materials to be used. In the Iowa practice the specifications for proportions are so written that the quality of concrete is predetermined by means of limiting water cement ratios, and the quantities of materials to be used are established within a very narrow range.

SPECIFICATION

PROPORTIONS FOR CONCRETE—SINGLE COURSE PAVEMENT IOWA HIGHWAY COMMISSION—1928

Mix no.	Proportion by dry wt.; lb. cement to lb. sand to lb. coarse aggregate			Max. total water allowable gal. per bag of cement	Max. size of aggregate allowable—inches Class 1, 2, or 3 aggregate	Approx. bbl. Cement per cu. yd. of concrete
1	1	1.71	3.47	5.5	1.5	1.63
2	1	1.93	2.91	5.5	2.5	1.71
3	1	2.06	2.53	5.5	2.5	1.78
4	1	2.17	2.17	5.5	2.5	1.85
5	1	2.25	1.85	5.5	2.5	1.92
6	1	2.30	1.54	5.5	2.5	2.00

" These proportions are based upon aggregates having specific gravities of 2.65. If materials are furnished which have specific gravities of more than 2.67 or less than 2.63, the proportions will be adjusted in accordance with the ratio which the actual specific gravity bears to 2.65.

" The estimated quantities of cement per cubic yard of concrete shown in the table of proportions are based upon average conditions. If the combination of materials is such that the quantity of cement used is consistently more than 1 per cent greater or more than 1 per cent less than the estimated quantity the proportions will be adjusted accordingly. This does not apply to variations in quantities caused by variation in the thickness of the slab."

The essential feature of this specification for proportions is the maintenance of constant absolute volumes of aggregates in each batch.

STRENGTH

In order to provide for some parts of the state in which the supply of fine aggregate greatly exceeds the coarse, a choice of six different base proportions is offered in the specifications. These mixtures where of similar materials and of the same consistency will yield concrete of equivalent strength. Bulletin No. 60 of the Engineering Experiment Station, Iowa State College, reports the experimental work upon which this system of proportioning is based. It is also true that if the same consistency is maintained in these various mixtures a constant water-cement ratio must be used. Therefore the minimum strength to be expected is controlled by a maximum water requirement in the specification.

QUANTITIES OF MATERIALS

An estimate of the quantity of cement needed per cubic yard of concrete is included in the specification in order to give a uniform basis for estimating bids.

There are two factors, due to the materials, which may in practice cause variation in the quantities of materials used. (a) Material of different specific gravity than that used in the estimate and (b) the production of concrete of different density than expected. With plastic mixes the density varies with the amount of mixing water. The quantities of materials needed for a cubic yard of concrete may be estimated as follows:

Proportion 1—1.71—3.47, using 5.5 gallons of water
per bag of cement. 6 bag batch.
Cement, 6 bags @ 94 = 564 lbs.
Fine aggregate = 964 lbs.
Coarse aggregate = 1957 lbs.

Assuming the specific gravity of cement to be 3.14 and of the aggregates to be 2.65, the volume of space that will be occupied by the particles of the various materials may be computed as follows:

One cubic foot of cement absolute volume will weigh 3.14 (specific gravity) × 62.5 lb. (wt. of cu. ft. of water) = 196.2 lbs. therefore the

space to be occupied by 564 lbs. $= \frac{564}{196.2} =$ 2.876 cu. ft.

and similarly the fine aggregate $= \frac{964}{2.65 \times 62.5} =$ 5.82 cu. ft.

and similarly the coarse aggregate $= \frac{1957}{2.65 \times 62.5} =$ 11.815 cu. ft.

water, 33.0 gals. = 4.40 cu. ft.

24.91 cu. ft. =

the yield of a 6 bag batch if there is enough water used so that there are no air voids.

Since 24.91 cu. ft. of concrete required 6 bags, the requirements for

one cubic yard is $\frac{6 \times 27}{24.91} =$ 6.50 bags = 1.63 bbl.

Any change in either of the two variables in the computation, specific gravity or water content will cause a change in the yield of the batch, and if the amount of cement per cubic yard is to be held within one per cent it is evident that occasional changes may have to be made in the proportions.

The specifications provide for making such changes after the job conditions are stabilized.

By using a constant water-cement ratio with the same cement, sand and coarse aggregate, the consistency will be constant for all the proportions given in the table. However, a change in the kind of materials used may require a change in the water-cement ratio in order to produce concrete of the consistency necessary for proper workability on the road. Since a change in the water-cement ratio will cause a corresponding change in yield, it is necessary to be prepared to make adjustments in mixtures if the yield of concrete for a given quantity of cement is to be kept constant.

Methods of Control

The objectives are (1) to produce uniform concrete of the required strength by maintaining a uniform water cement ratio after same has been established for proper workability on the job and by uniform measurement of the other materials in the required proportion, and (2) to use such quantities of materials as will produce concrete containing an amount of cement within one per cent of that estimated in the specifications.

The materials are all inspected before arrival at the site of the work and a record of the data on each carload is kept.

The duties of the proportioning plant inspector are: To supervise storage of materials, especially the building of the stock piles in order to minimize segregation, to see that the proper quantities of materials enter each batch, and to test the specimens of concrete sent in from the pavement. Supervision of measurement involves determination of specific gravity, and the moisture content of the aggregates, since the proportions are by dry weight.

Beam specimens for use in determining time of opening of pavement, and for general record as to quality are made by the inspector in charge at the mixer. This inspector also supervises the measurement of water. His other duties have to do with the placement of the concrete, with which this paper is not concerned.

Adjustments of proportions other than for specific gravity or moisture are made by the district material inspector or higher authority.

The proportioning plant inspector is provided with laboratory equipment for the performance of the necessary tests and a small wooden building is furnished for his use, as shown in Figures 1 and 2. The cost of the small building is about $50.00 and of the equipment about $100.00.

SPECIFIC GRAVITY AND MOISTURE IN AGGREGATES

It is necessary to know the specific gravity of the materials, both for use in adjusting proportions, and for use in determining the moisture carried by the material or that absorbed.

The proportions are arranged fundamentally on the basis of absolute volumes, but are stated in the specifications by weight of aggre-

Figure 1. Field Laboratory

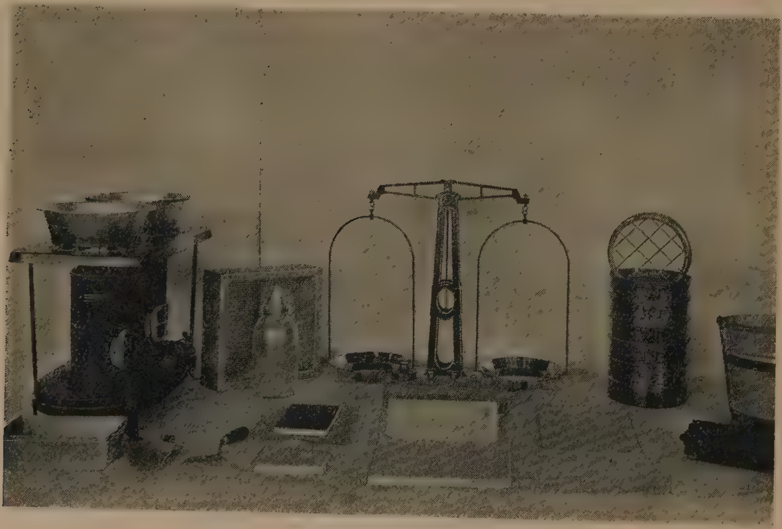

Figure 2. Field Laboratory Equipment

gates having specific gravities of 2.65. That is, so many pounds of cement to so many pounds of sand having a specific gravity of 2.65 when in a saturated but surface dry condition to so many pounds of coarse aggregate having a specific gravity of 2.65 in the saturated, surface dry condition. The specific gravity is first determined for the materials to be used, in this surface dry but saturated condition. One determination will usually suffice, so long as the source of supply is not changed. A simple form of pycnometer or measuring bottle is used for determining either apparent specific gravity, free water in the sample or the water absorbed by the particles of the sample. This pycnometer consists of a common fruit jar with a conical top, as shown in Figure 3. Figure 4 shows the type of balance needed for this work.

DETERMINATION OF SPECIFIC GRAVITY

The specific gravity of the aggregate particles in a saturated surface dry condition is determined as follows:

(a) *Coarse Aggregate.* The apparent specific gravity of coarse material shall be determined upon material that has been immersed in water at 70° F. for 24 hours. The pieces shall be individually surface dried with a towel. Two thousand grams of this surface dried material shall be weighed out to the nearest 0.5 gram. This sample shall be placed in the pycnometer and the pycnometer partly filled with water at 70° F. The pycnometer shall be gently shaken and rolled to remove entrained air. The pycnometer shall then be exactly filled with water and its outside surface dried. The pycnometer and its contents shall then be weighed to the nearest 0.5 gram. The apparent specific gravity shall be computed from the formula.

$$\text{Specific gravity} = \frac{S}{P + S - W}$$

where

S = the weight of the sample in grams.
P = the weight in grams of the pycnometer full of water.
W = the weight in grams of the pycnometer plus the sample plus sufficient water to fill the pycnometer.

(b) *Fine Aggregate.* The apparent specific gravity of fine material shall be determined in the same way as for coarse material except that the sample shall consist of air dried material and shall consist of 1000 grams.

For making changes in the weight proportions on account of specific gravity, the saturated, surface dry determination applies exactly to

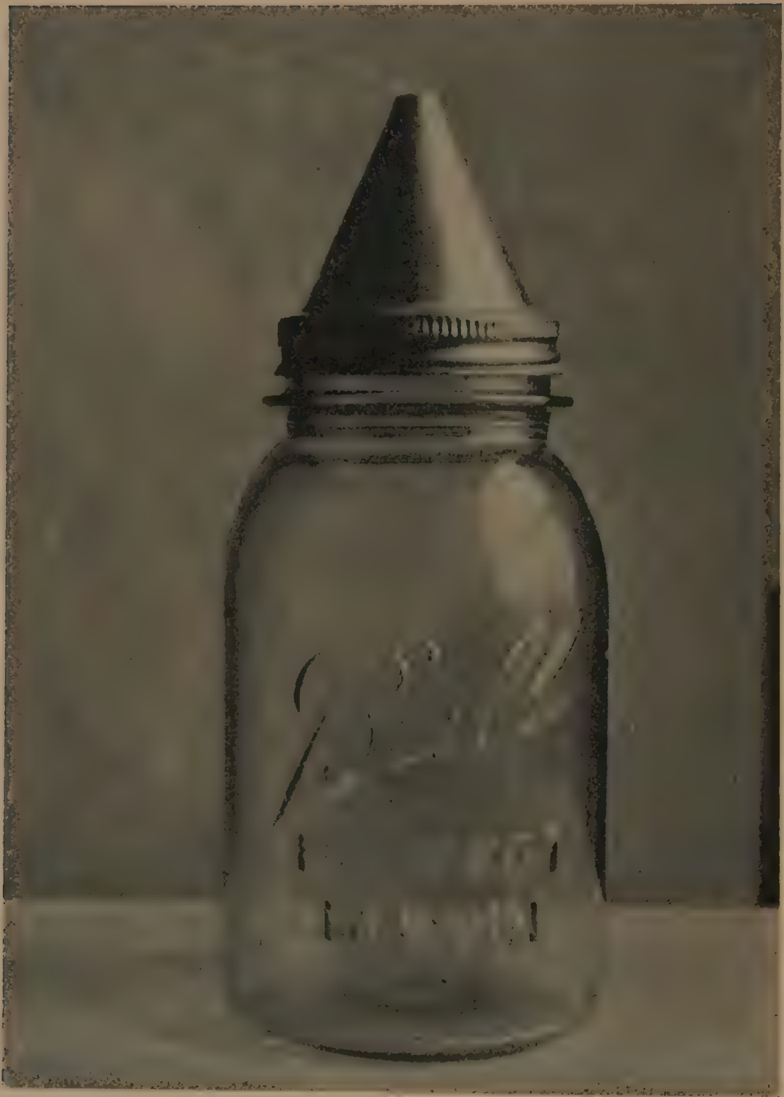

Figure 3. Pycnometer for Determination of Apparent Specific Gravity, Contained Moisture and Absorption of Aggregates

materials which reach the job carrying some free water. To be exact in the case of materials which do not carry free water but absorb water during the mixing and placing operations, the specific gravity used should be that of the material containing only the moisture that would be absorbed during the mixing and placing period. However the error being small the specific gravity saturated and surface dry has been used in practice. This is a point susceptible of further refinement in the case of very porous materials.

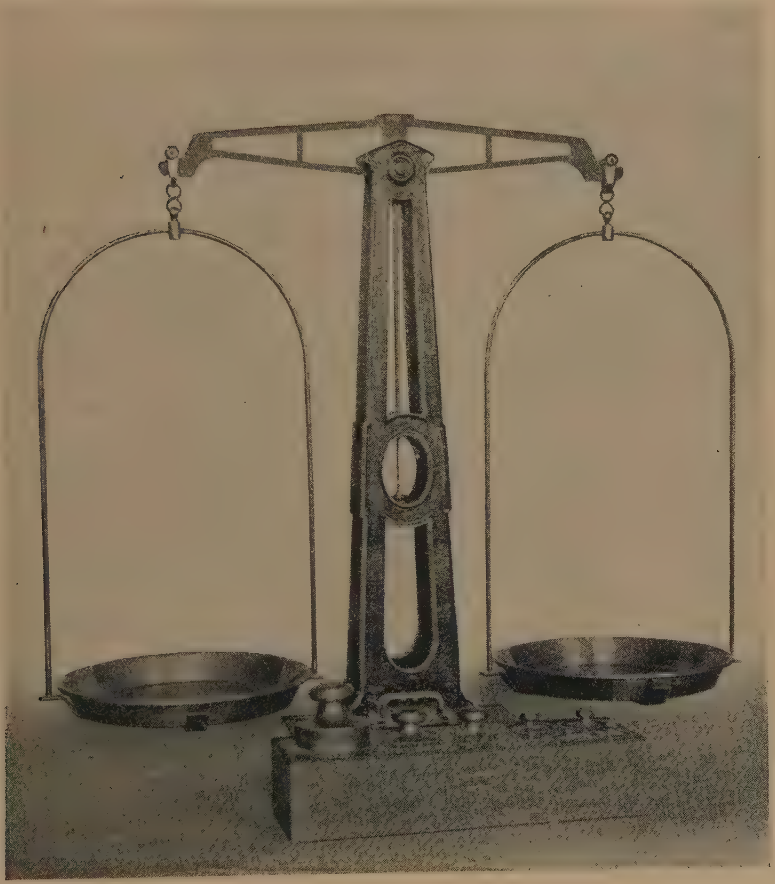

Figure 4. Balance for Use in Determining Apparent Specific Gravity, Contained Moisture, and Absorption of Aggregates

DETERMINATION OF MOISTURE IN AGGREGATES

Since aggregates are seldom saturated and surface dry it is necessary to adjust the batch weights to compensate for the free moisture present or for the moisture to be absored by the aggregates as the case may be. The same apparatus may be used for determining the free moisture or absorption as for specific gravity. The theory for the use of the pycnometer method is as follows:

Let $P =$ Weight of pycnometer filled with water, in grams.

Let $G_s =$ Specific gravity of material when in a saturated surface dry condition.

Let $s =$ Weight of sample in grams.

Then $\frac{s}{G_s} =$ The volume of the sample if it were in a saturated surface dry condition, in cubic centimeters.

Let $W = P + s - \frac{s}{G_s}$ which is the weight of the pycnometer plus weight of the sample plus the weight of enough water to fill the pycnometer with the sample in the pycnometer, if the sample were saturated and surface dry.

Let $x =$ the number of grams of free water in the sample or the number of grams of water that will be absorbed by the sample (x will be positive if the sample contains free water and negative if the sample absorbs water).

Then $\frac{s - x}{G_s} + x =$ The volume which will be displaced by a sample of weight s, but containing $+ x$ grams of free moisture.

Let $W_1 = P + s - \frac{(s - x)}{G_s}$ which is the weight of the pycnometer plus the weight of a sample containing $+ x$ grams of free moisture plus enough water to fill the pycnometer with this sample in the jar.

$$W - W_1 = P + s - \frac{s}{G_s} - P - s + \frac{(s - x + x)}{G_s}$$

$$W - W_1 = \frac{s}{G_s} + \frac{s - x}{G_s} + x$$

$$W - W_1 = - s + s - x + x\,G_s = x\frac{(G_s - 1)}{G_s}$$

$$x = (W - W_1)\frac{G_s}{G_s - 1}$$

If the sample contains free water in the amount x, the amount of saturated, surface dry aggregate in the wt. of sample s will be less than if the sample is just in the saturated surface dry condition, therefore the weight W_1 will be less than the weight W and x will be positive. If on the other hand the particles of the sample are in condition to absorb moisture, W_1 will be greater than W and therefore x will be negative.

The manner of conducting the test is as follows:

The weight of the pycnometer filled with water at 70° F. (P) is determined. This will remain constant for any given pycnometer. The size of sample having been decided upon and G_s having been determined W can be computed. This quantity will be a constant for any given material for a given pycnometer and a given size of sample.

A sample representing the material to be used and of the quantity previously decided upon is weighed out. This sample is placed in the jar of the pycnometer, which is partly filled with water. The jar is then filled with water, and is rolled and shaken to remove any entrained air. Water is then introduced through the hole in the cap until the pycnometer is full. All the water used shall be at 70° F. Any excess water on the outside of the pycnometer is wiped off and the jar and contents is weighed. This weight is W_1.

$$x = (W - W_1)\,\frac{G_s}{G_s - 1}$$

is then computed. It will be noted that for the conditions mentioned above all the quantities in the formula are constants except W_1.

Having determined the number of grams of water in the sample, the following computation is made.

$\frac{100\,x}{s}$ = the per cent of the sample taken which is free moisture or the per cent water absorbed based on the original weight of the sample.

Since the moisture determination can be made in about ten minutes the inspector can easily make a sufficient number to insure that he has detected all variations that may affect the batches. The variation in moisture will not be large for materials that have been in storage piles for any length of time, but there may be considerable variation from car to car or from car to stock pile.

The inspector is required to make a sufficient number of tests to insure that he has detected all variations that may affect the material for any considerable period of time. One moisture test for each material every two hours is considered a minimum.

The largest variations usually occur in the fine aggregate. The most uniform condition prevails when the materials are handled through a large stock pile. The moisture content of materials in cars varies from car to car and from place to place in the same car. These variations are affected by the condition of the cars and by the hauling distance from the loading plant.

TABLE I

Data on Moisture in Sand from 94 Jobs

Project no.		Percentage of moisture			
		Min.	Max.	Average	Variation per cent
81	Job showing highest average moisture content	3.7	6.6	4.9	2.9
11	Job showing lowest average moisture content	0.8	2.6	1.6	1.8
63	Job showing greatest variation in moisture content	1.9	7.9	4.2	6.0
69	Job showing least variation in moisture content	3.0	3.5	3.2	0.5

TABLE II

Data on Moisture in Gravel from 50 Jobs

Project no.		Percentage of moisture			
		Min.	Max.	Average	Variation per cent
85	Job showing highest average moisture content	1.4	1.6	1.4	0.2
31	Job showing lowest average moisture content	−1.2	1.0	−0.1	2.2
26A	Job showing greatest variation in moisture content	−1.8	4.7	0.2	6.5
30	Job showing least variation in moisture content	0.0	0.3	0.1	0.3

The minus sign indicates that the material absorbed water instead of carrying free moisture.

TABLE III

Data on Moisture in Limestone from 45 Jobs

Project no.		Percentage of moisture			
		Min.	Max.	Average	Variation per cent
91	Job showing highest average moisture content	0.0	1.6	0.9	1.6
80	Job showing lowest average moisture content	−1.6	0.5	0.7	2.1
19	Job showing greatest variation in moisture content	−2.1	3.0	0.5	5.1
88	Job showing least variation in moisture content	−0.5	0.1	−0.4	0.6

A typical example of the hour to hour variations is shown in Figure 5.

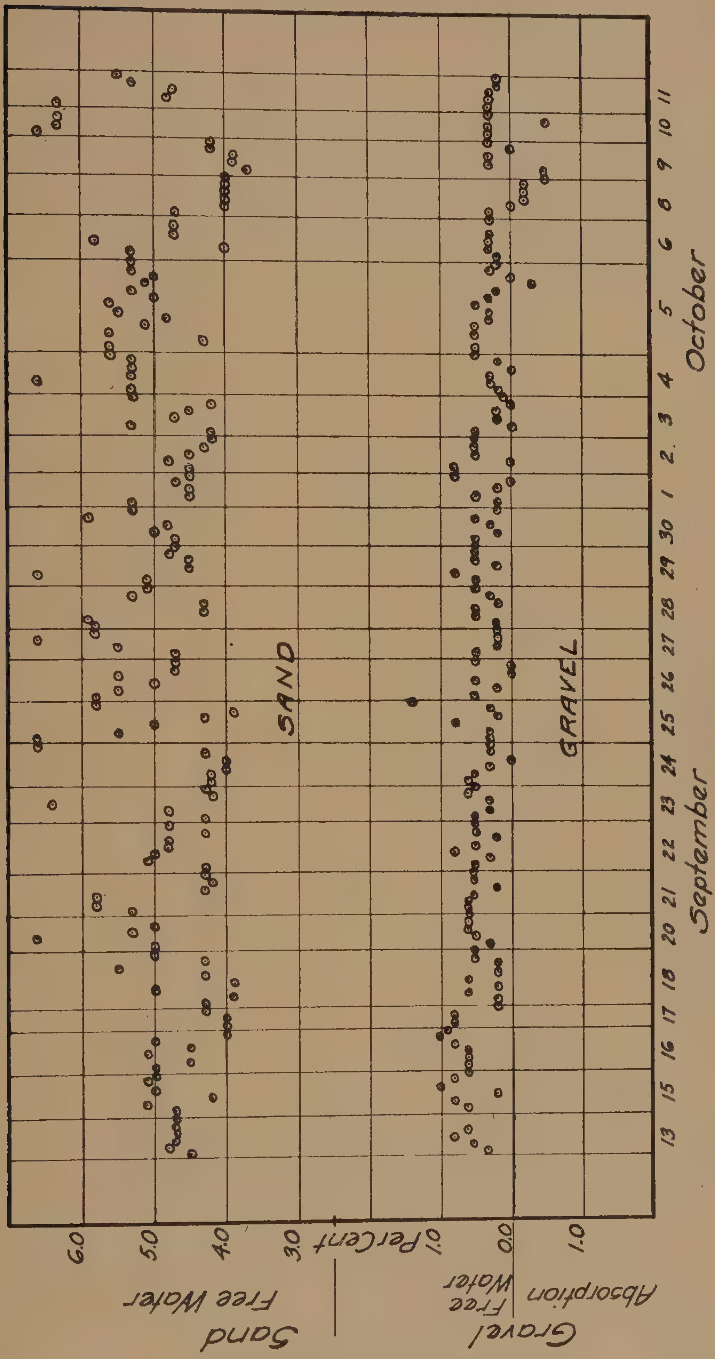

Figure 5. Day to Day Variation in Moisture Condition of Aggregates. September 13 to October 11—Project 81—Iowa Highway Commission

The range in moisture conditions encountered from job to job is shown in Figures 6 and 7.

When it is remembered that a variation in moisture of 3 per cent of the weight of fine aggregate may mean a variation of 0.6 gallon per bag of cement in the water cement ratio of the batch, and twice as much for an equal variation in the coarse aggregate it appears that a determined effort is justified in making correction for these variations. A variation of 0.6 gallon per bag of cement is equivalent to a change in strength of about 10 per cent for the data shown in Figures 18 and 19.

BATCH WEIGHTS

The inspector having determined the specific gravity of the aggregates, the percentage of moisture, the percentage of sand in the coarse aggregate, and the percentage of coarse aggregate in the sand computes the batch weights as illustrated by the following example:
Assume—

Specified proportion—1—1.93—2.91 weight	
Specific gravity of fine aggregate	2.65
Specific gravity of coarse aggregate	2.55
Per cent passing the No. 4 sieve—fine aggregate	85.00
Per cent passing the No. 4 sieve—coarse aggregate	4.00
Per cent moisture—fine aggregate	4.50
Per cent moisture—coarse aggregate	2.30
Size of batch—7 bags of cement	

Correction for Specific Gravity. Since the specified proportion is based upon specific gravities of 2.65, a proportion which would give the same absolute volumes will be obtained by multiplying the aggregate parts by the ratio of the specific gravity of the material to be used to 2.65 thus.

$$1 - \frac{2.65}{2.65} \times 1.93 - \frac{2.55}{2.65} \times 2.91 = 1 - 1.93 - 2.80$$

Correction for Sand in Coarse Aggregate and Coarse Aggregate in the Sand. The specified dividing line between fine and coarse aggregate is the No. 4 sieve. A correction is not made unless one or the other of the materials contains more than 5 per cent over or under size, since the specifications allow a tolerance of 5 per cent on the No. 4 sieve. In the example the fine aggregates contain 15 per cent coarser than the No. 4 sieve so the correction will be made.

Proportion to maintain 1 : 1.93 : 2.80.

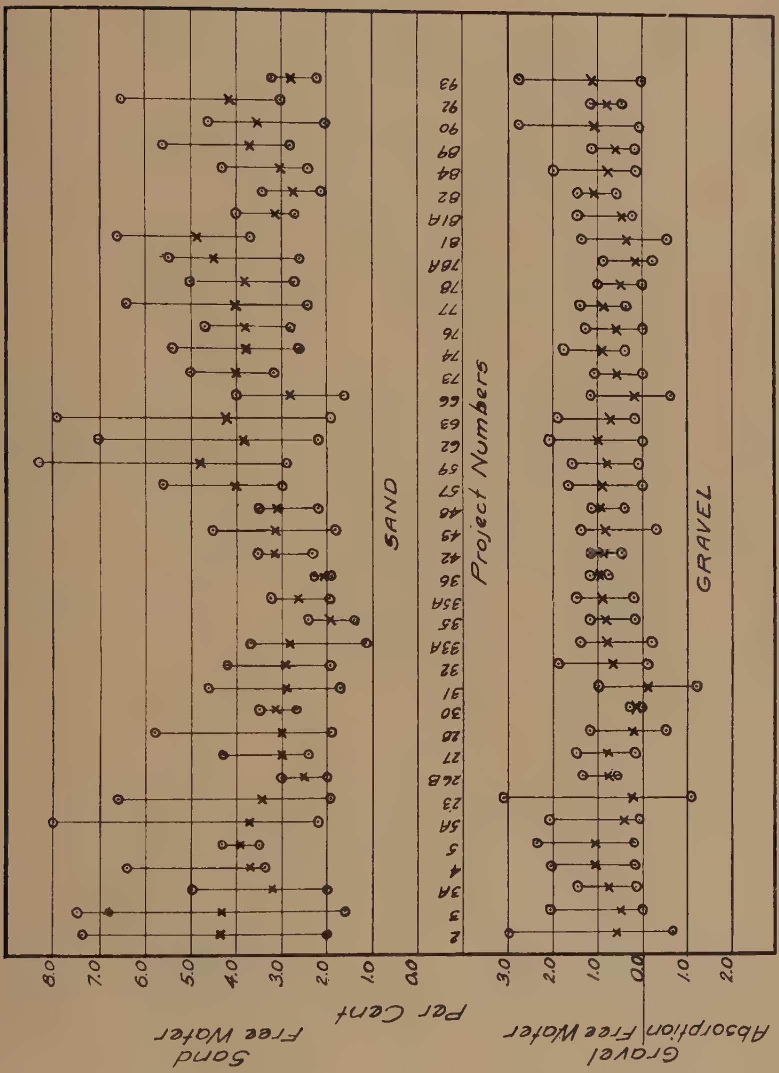

Figure 6. Maximum, Minimum and Average Moisture Content of Aggregates. Gravel Aggregate Projects

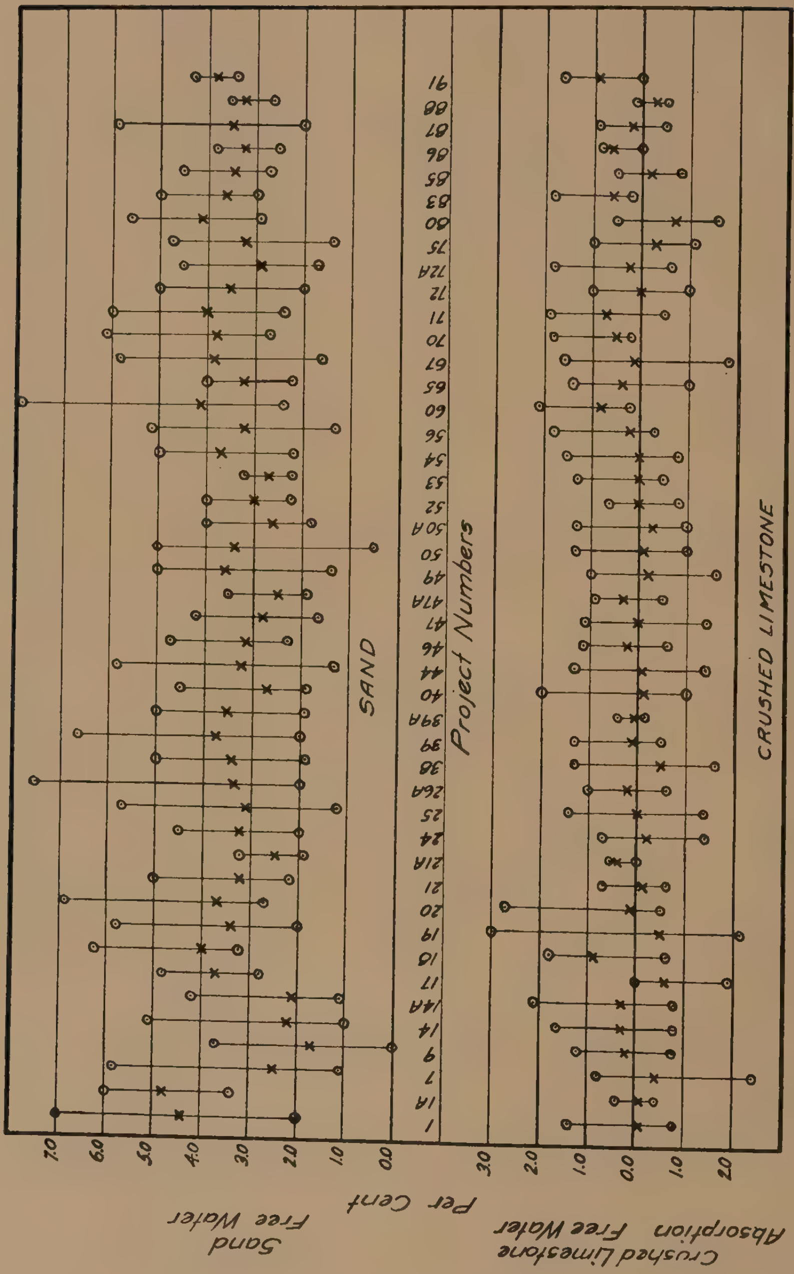

Figure 7. Maximum, Minimum and Average Moisture Content of Aggregates. Crushed Limestone Aggregate Projects

Since the specific gravities of the two materials are different, the correction can best be made by reducing the proportion to absolute volumes. Using a batch containing 196 lbs. of cement the absolute volume relation will be as follows: To determine the absolute volume of any weight of a given material, divide the weight by the specific gravity of the material times the weight of one cubic foot of water.

Cement 196 lbs., $\frac{196}{3.14 \times 62.5} = 1.00$ cu. ft. absolute volume.

Sand 378 lbs., $\frac{378}{2.65 \times 62.5} = 2.28$ cu. ft. absolute volume.

Coarse Aggregate 549 lbs., $\frac{549}{2.55 \times 62.5} = 3.31$ cu. ft. absolute volume.

The proportion by absolute volume is therefore 1 : 2.28 : 3.31.

Let x = corrected absolute volume of fine aggregate to use as furnished and y = corrected absolute volume of coarse aggregate to use as furnished, then

$$x + y = 2.28 + 3.31 = 5.59$$

and

$$.85\,x + .04\,y = 2.28.$$

Solving

$$x = 2.54$$
$$y = 3.05.$$

Therefore the corrected mix by absolute volumes will be 1 cu. ft. cement to 2.53 cu. ft. of the fine aggregate at hand to 3.05 of the coarse aggregate.

To find the weight of a given absolute volume of material, multiply the volume by the specific gravity times the weight of a unit volume of water. The corrected weight relationships will be therefore:

Cement—$1 \times (3.14 \times 62.5$ lbs.$) = 196$ lbs.

Fine aggregate—$2.53 \times (2.65 \times 62.5$ lbs.$) = 418$ lbs.

Coarse aggregate—$3.05 \times (2.55 \times 62.5$ lbs.$) = 486$ lbs. and the corrected proportion is 1 : 2.13 : 2.48.

Correction for Moisture. The fine aggregate contains 4.5 per cent of moisture.

Let x = amount of fine aggregate and moisture needed to produce 2.13 parts of fine aggregate

then

and $$.955\,x = 2.13$$

$$x = 2.23$$

The coarse aggregate contains 2.3 per cent moisture.

Let y = amount of coarse aggregate and moisture to produce 2.48 parts of coarse aggregate

then

and $$.977\,y = 2.48$$

$$y = 2.54$$

Therefore, having made the adjustment for specific gravity, over and under size, and moisture, in order to produce concrete equivalent to the base proportion the materials at hand will need to be measured in the proportion of 1 : 2.23 : 2.54.

A 7-bag batch will require:

7 × 94 × 2.23 = 1467 lbs. fine aggregate as furnished

7 × 94 × 2.54 = 1671 lbs. coarse aggregate as furnished

The computations are very much simplified in practice by means of tables and diagrams.

The total net amount of moisture in the aggregates per batch is communicated from the plant inspector to the inspector at the mixer so that the amount of water added at the mixer can be regulated in accordance therewith.

MEASUREMENT OF AGGREGATES

Aggregates are measured by weight. As weighing has been used on all paving work under the Iowa Highway Commission for five years, the methods and appliances have become well established on a practicable basis. The details that have been found to be important have been crystallized in the following specifications:

" Equipment for weighing aggregates shall be capable of weighing accurately to two (2) pounds and shall comply with the following conditions:

" 1. At least that part of the total load weighed which is a fraction of one hundred pounds shall be indicated on a graduated beam or dial.

"2. The weighing equipment must be so arranged that the operator stands, when operating the bin gates, at such a position that he can conveniently shovel material from the weighing hopper. The weighing beam or dial shall be in full view of the operator when he is in a position to operate the gate which delivers materials to the weighing hopper.

"3. There must be sufficient clearance between the top of the weighing hopper and any other part of the bin structure that the scale operator can conveniently shovel material from the weighing hopper.

"4. In the case of weighing hoppers mounted on platform scales, the arrangement shall be such that the center of gravity of the total load on the scale is in the same vertical line as the center of the scale platform."

The following instructions to the inspectors in charge of the weighing plants are the result of five years of experience.

INSPECTION OF WEIGHING DEVICES

"Before concrete operations are started at any installation, the bins shall be fully loaded and allowed to stand for at least twelve hours. The inspector shall then check the accuracy and the sensitiveness of each weighing device used to the full amount of any batch that will be used. The contractor is required to provide 400 pounds of standard fifty-pound weights for each scale installation.

"The scale shall first be balanced with no load. The standard weights shall then be placed on the scale in such a way as to apply the load at the same position as the load will be applied by the materials to be weighed.

"The standard weights shall be applied one at a time and the scale reading for each increment of load recorded in a permanent notebook. When the last standard weight has been applied, the weights shall be removed without changing the scale setting. The scale shall then be accurately balanced by placing material in the weighing hopper. The standard weights shall then be added to this weight as before and this process repeated until the required capacity is reached. The sensitiveness of the scale shall be checked at each increment of load within two hundred pounds of the required capacity by sliding the poise each way until the beam rises or falls as the case may be. The required movement each way shall be recorded.

"If the scale is not found to be sensitive to two pounds, the scale shall not be used until it is repaired or adjusted. If the scale is not accurate to five pounds, it shall not be used until it is repaired or adjusted except when the scale is found to be sufficiently sensitive and the calibration curve is a straight line. In such a case the scale may be used for the period necessary to have proper repairs or adjustments made. When a scale is used under these circumstances, the correction necessary to compensate for the error in the scale shall be made.

"The inspector shall inspect the scales frequently to detect any sign of sluggishness, inaccuracy or damage.

"The scales shall be checked fully for accuracy at intervals of one week or less or at any time that accident or settlement may have affected them.

"The scales shall be balanced under no load at least twice each day of work. At least once each working day the scales shall be checked for accuracy and sensitiveness by applying one standard fifty-pound weight to the full load and the indicated difference in weight and the sensitiveness observed. If the scales have long exposed levers, the contractor shall be required to provide the scales with suitable protection from the wind.

INSPECTION OF WEIGHING

"One of the most important duties of the inspector is the inspection of the weighing of the aggregates. He should spend as much time as possible observing this operation. If the proper inspection of the addition of cement to the batch requires the inspector to be away from the scales at the time when aggregates are being weighed, an additional inspector shall be arranged for by the resident engineer.

"The scales, set at the required weight, shall be accurately balanced by the load for each batch.

"In most cases, it will be found impractical to attempt to weigh out a batch at a single opening of the bin gate. It will ordinarily be necessary to set the scales for some weight less than the weight required for the batch. The gate is then opened and left open until the scale beam rises. The gate is then closed and the scale set at the required weight. The gate is then partially opened and material allowed to flow slowly into the weighing hopper until the scale is balanced.

"If an overload has been applied, it must be removed.

"The inspector must insist upon extreme care on the part of the scale operator. Repeated cases of carelessness in this respect is sufficient cause for having the scale operator removed.

"Equipment for the satisfactory weighing of aggregates has been arranged in a variety of ways. Figure 8 shows the simplest outfit used. This consists of a wooden hopper resting on the platform of an ordinary platform scale. Figure 9 shows a typical set-up of permanent equipment especially designed for this purpose."

MEASUREMENT OF WATER

Accuracy of water measurement comparable to that secured with other materials has not yet been attained. The equipment furnished with the newer concrete mixers, although much improved and accurate under some conditions still does not insure constant measurement from batch to batch on a pavement mixer.

Mr. Bert Myers, Engineer of Materials, Iowa Highway Commission, discussed some experiments on the customary water measuring devices at the 1928 Highway Research Board Meeting.

These experiments include measurements by water meter upon seven mixers of three makes, for one-half day each. Mr. Myers concludes that most of the water measuring equipment of the type which consists of a closed tank discharging into the mixer drum through an adjustable syphon, trough, pipe or displacement cylinder designed to allow a controlled quantity of water to be drawn from the tank, will measure water accurately when the supply has a uniform pressure and contains little entrained air. However, in practice where pressures varied in short periods of time as much as 150 pounds per square inch, and opportunities existed for entrance of air, he found errors in measurement in all cases ranging from 2.50 gallons per batch to 7.28 gallons per batch. It was found that the valve arrangement which permits the passage of water not measured in the tank, when the valves are operated was largely responsible for the excess of water. As this excess is directly proportional to the supply line pressure the variation is correspondingly large.

"It appears from these observations that the present method of water measurement employed on paving mixers is too inaccurate to give the control of the quality of concrete that is desired and that the difficulties encountered in the field are such that accurate measurement of water by volume in a closed tank is not to be expected. Therefore, some new method of water measurement seems to be desirable. Measurement in a tank open to the air would reduce the errors due to the presence of entrained air in the pipe line and would reduce the leakage through the valves since the pressure on the valves would be

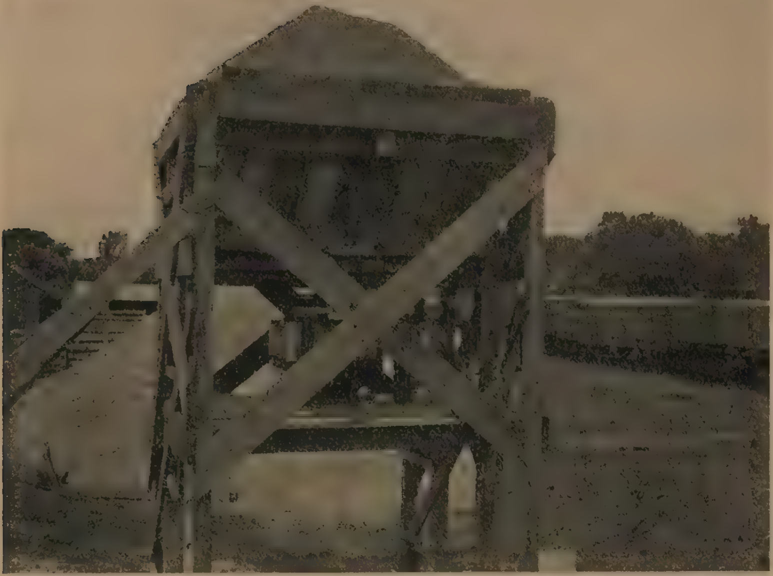

Figure 8. Plant for Weighing Aggregates. Wooden Hopper on Platform Scales

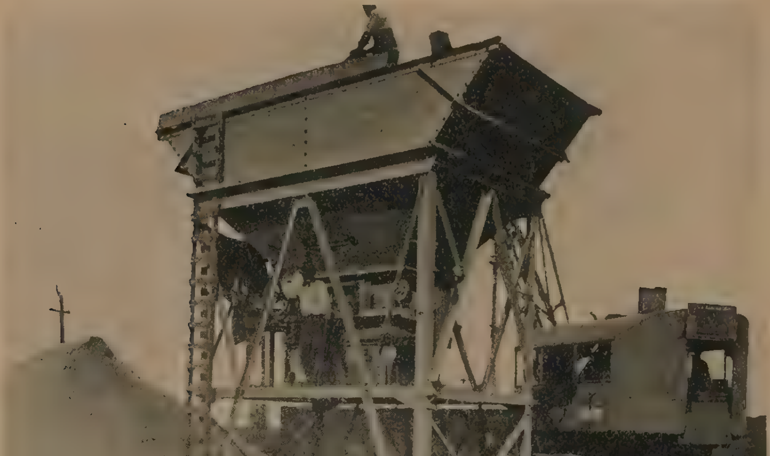

Figure 9. Steel Bins, Hopper and Weighing Arrangement for Aggregates

greatly reduced. By the use of an open tank, some of the simplicity in arrangement and operation possible when a closed tank is used would be sacrificed."

In an attempt to eliminate some of the errors prevalent in water measurement the specifications of the Iowa Highway Commission now contain the following requirements:

"The water measuring equipment shall be capable of accurate measurement to one quart (¼ gallon) and shall be so arranged that the accuracy of measurement will not be affected by variations in pressure in the water supply line. Unless the water is to be weighed the water measuring equipment shall include an auxiliary tank from which the measuring tank shall be filled. The volume of the auxiliary tank shall be at least equal to that of the measuring tank. The equipment shall be so arranged that the water pressure in the measuring tank cannot exceed that due to the difference in elevation between the two tanks."

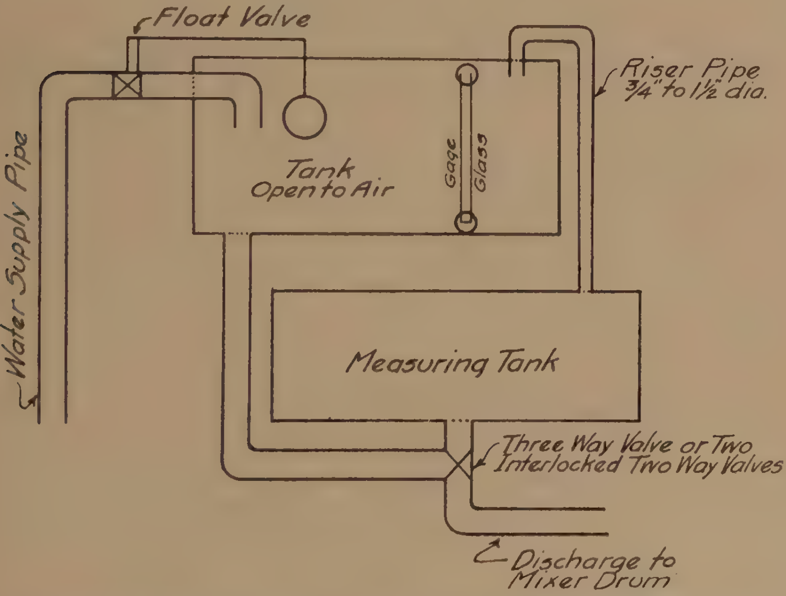
Float Valve
Riser Pipe 3/4" to 1½ dia.
Water Supply Pipe
Tank Open to Air
Gage Glass
Measuring Tank
Three Way Valve or Two Interlocked Two Way Valves
Discharge to Mixer Drum

Figure 10. Diagrammatic Sketch of Water Measuring Equipment on Paving Mixer to Comply with Iowa State Highway Commission Specification

Figure 10 shows a diagrammatic arrangement of tanks to comply with this specification.

ADJUSTMENT OF PROPORTIONS

On each project as soon as possible after conditions are stabilized, the actual volume of concrete laid for a run of four hundred feet or more is carefully measured, in order to compare the quantities of materials being used with the estimated quantities shown in the specifications. If the quantity of cement used per cubic yard of concrete varies more than 1 per cent from the estimate, the proportions are changed enough to correct this condition. This survey also gives information as to the thickness of slab being laid, to compare with the plan thickness. A typical case was handled as follows:

The volume of concrete laid in four hundred feet of pavement was determined from measurement of the subgrade before the concrete was placed and of the surface of the concrete to get accurate cross sections of the slab at 10-foot intervals. Measurements across the slab were taken every two feet.

Length of section measured = 400.0 feet
Measured volume = 4741.99 cu. ft.
Plan volume = 4617.54 cu. ft.
Average excess thickness = 0.21 inches
Overrun in cement due to excess thickness = 2.80 per cent

According to the inspector's report: "These data show the slab to be more than the required thickness at all points except the west edge where about $\frac{1}{8}$ of an inch form settlement was found. The form settlement on the east side was only 3/32 inch." The proportions of the concrete are not changed to correct for the overrun due to this use of excess concrete, but the information is of value to the inspector and to the contractor's foreman.

Theoretical cement for 400 feet of correct cross section
= 1216 bags
Cement used = 1276.26 bags
Total overrun = 4.90 per cent
Overrun due to combination of materials
= 4.9 — 2.8 = 2.1 per cent

or computed in another way:

Cement estimated = 1.78 bbl. per cubic yard
Measured volume of concrete = 4741.99 cubic feet = 175.63 cu. yds.
Cement used = 319.065 bbl.

Cement used per cubic yard $\frac{319.065}{175.63} = 1.817$ bbl.

Overrun due to materials = 2.1%

The use of more cement than estimated was probably due to the use of a lower water cement ratio than estimated. If an accurate record of the amount of water used were available the proportions could be easily and definitely corrected by designing a mix with the new water cement ratio to use the estimated amount of cement. However, as shown in the discussion on water measurement, these data are not available. A comparison of the yield of a one-bag batch as placed will indicate the correction to be made.

Cubic feet of concrete per bag of cement:

$$\begin{array}{ll} \text{Estimated} = 3.791 \\ \text{Actual} = 3.715 \\ \hline \quad .076 \text{ cu. ft.} \end{array}$$

Assuming that a small amount of aggregate can be added to the batch without an increase in the water per batch, the mixture can be corrected by adding to each batch .076 cu. ft. of aggregate (absolute volume).

As the aggregates in the original mixture were in the ratio of 45 to 55, 45 per cent of the addition should be sand and 55 per cent gravel.

$$45\% \text{ of } .076 \text{ cu. ft.} = 0.034 \text{ cu. ft.}$$
$$55\% \text{ of } .076 \text{ cu. ft.} = .042 \text{ cu. ft.}$$

The specific gravity of the sand and gravel is 2.65. The weight per cu. ft. of sand or gravel absolute volume $= 2.65 \times 62.5 = 165.60$ lbs. Therefore the weights of aggregates to add to each batch are:

$$\text{Sand} = .034 \times 165.6 = 5.63 \text{ lbs.}$$
$$\text{Gravel} = .042 \times 165.6 = 6.95 \text{ lbs.}$$

BATCH DATA—ONE BAG BATCH

	Original	New
Cement	94 lb.	94
Sand	193.64 lb.	199.27
Gravel	237.82 lb.	244.77

New proportion = 1 — 2.12 — 2.60

As a matter of fact it is probable that for the same workability the new batch will need a small addition of water, which will have an effect upon the quantities. However, experience has shown that the result will still be within the 1 per cent variation allowed by the specification. It is doubtful, field conditions being what they are, that closer control than 1 per cent is practicable.

The problems in design of mixtures and adjustment of proportions are capable of mathematical solution, except for one factor. That is workability. At present there is no known way of telling exactly what the workability of certain mixtures will be nor the effects of changes. This leaves the final adjustment on a cut and try basis as in the foregoing example.

FIELD TESTS

TIME OF OPENING PAVEMENT

Test specimens are made on each job to determine when the pavement has acquired sufficient strength for opening, and to afford research data. Two beams 6 by 6 by 30 inches are made each day. On the day following that on which the specimens are made and when they are 12 to 18 hours old they are moved with the molds in place to the plant inspector's laboratory and stored in damp sand until tested. Beams are tested normally at ages of 4 and 10 days, each beam being broken twice.

These specimens are tested as simple beams in a machine furnished the inspector for that purpose. The requirements for the testing machine are as follows:

" The following principles should be secured in the design of the particular apparatus that may be used to make transverse tests of concrete beams whether they are field or laboratory tests:

"(a) The distance between supports and points of load application shall remain constant for a given piece of apparatus.

"(b) The load shall be applied normal to the initial position of the neutral plane of the beam and in such a manner as to avoid eccentricity of loading.

"(c) The direction of reaction shall, at all times throughout a given test, be parallel to the direction of the applied load. If this be vertical, then the direction of the reactions shall be vertical and shall remain so throughout the test until rupture occurs.

" NOTE.—The more common devices, which, either single or in combination, essentially secure such a relation between the applied loads and consequent reactions, include link supports, rocker bearings, spherical bearings or flexure plates.

"(d) The application of the load shall be such as to secure uniform increments of load for equal time periods. This operation requirement demands that the operation of the apparatus be " smooth " and free from sudden shocks.

"(e) The ratio of distance between point of load application and the nearest reaction to depth of beam shall be not less than $1\frac{1}{2}$ to 1.

"NOTE.—The effect of this ratio on modulus of rupture requires further investigation."

This testing machine is illustrated in Figures 11 and 12. An interesting feature of this machine is the arrangement for application of the load to the specimen at a uniform rate of speed. Water is run through a flexible tube at a uniform rate, under practically constant head into a bucket suspended from the end of the scale beam. The load is applied to the specimen by means of the hand-crank and reduction gears at such speed as will keep the scale beam in poise, thus assuring uniform load application. When the beam breaks the water is automatically shut off. The container and water are then weighed, as shown in Figure 12, and the weight multiplied by the lever ratio of the machine to determine the breaking load. These machines are moderate in cost and have proven satisfactory in use.

One broken end from each beam is shipped to the headquarters laboratory, where the ragged end is sawed square, both ends capped and a compression test made at 28 days of age. The stone saw used for the purpose is illustrated in Figure 13.

RESULTS

The results secured by the system of control described herein may be measured by the record as to quantities of materials used, and the strength data secured from the test specimens. Data are reported from 69 projects covering 634 miles of pavement construction. 3754 transverse and 3303 compression tests were made. Table No. IV gives the average data for the various projects.

QUANTITIES OF MATERIALS

As evidence of the success of the attempt to control quantities of materials within reasonable limits may be cited the fact that for the first time in many years there were no controversies over quantities between the contractors and engineers. This is the more significant because the program for the year was over twice as large as the biggest year heretofore. The average overrun in cement used for all jobs was 1.2 per cent. The methods used for forming the sub-grade under the Iowa Specifications are such that some extra depth is practically inevitable, and when it is remembered that on a 7-inch slab an ex-

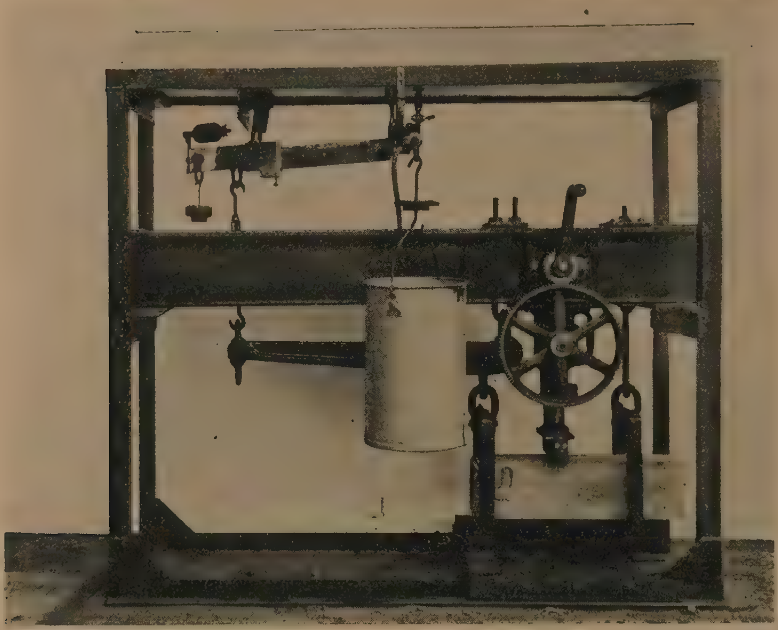

Figure 11. Transverse Testing Machine for Concrete Beams, Showing Use of Water to Keep Poise in Balance as Load is Applied. Water Used Instead of Weights, to Insure Load Application at a Uniform Rate

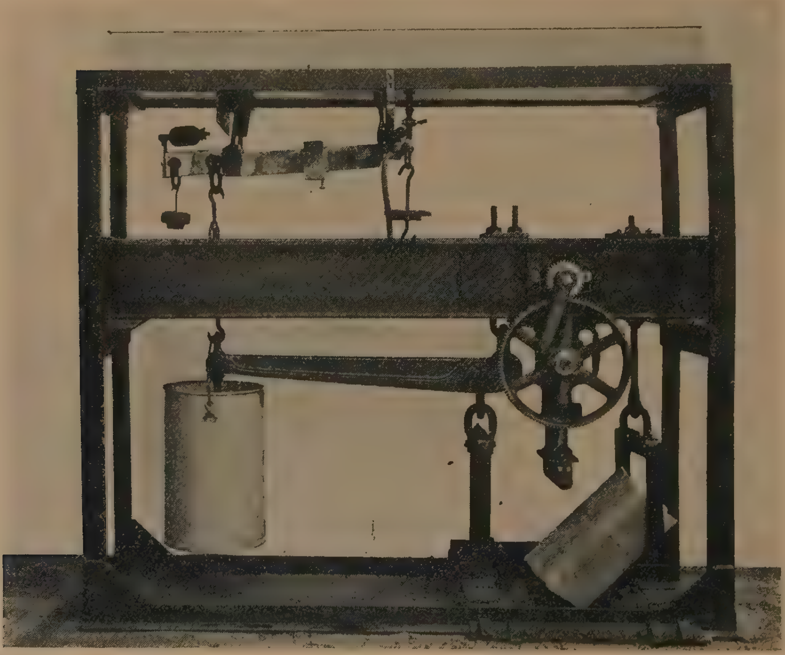

Figure 12. Transverse Testing Machine for Concrete Beams, Showing Method of Weighing the Water Used in Balancing the Poise

tra depth of only .07 inch will account for a 1 per cent overrun in materials, it is easy to believe that overruns due to the combinations of materials were practically eliminated. When a job continued after investigation and adjustment to show the use of cement in excess of the estimate it was always possible to show from the measurements of the slab that the fault lay in the use of excess concrete. Frequently extra concrete is used deliberately in order to avoid other more expensive difficulties. A comparison of the record under this specification with the work done in 1920 to 1923 under volumetric measurements is shown in Figure 14.

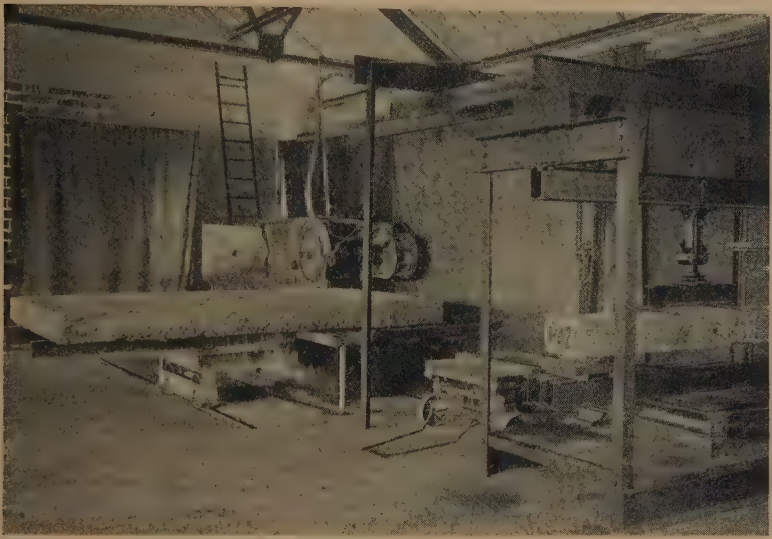

Figure 13. Stone for Trimming Specimens of Concrete

STRENGTH TESTS

In addition to the modulus of rupture tests, one end of each beam was sent to the laboratory, the broken end sawed square, and both ends capped with plaster for a compression test. The average results for each job are also shown in Table IV. In so far as strength is concerned the measure of the effectiveness of the control methods may be seen in the degree of uniformity attained.

UNIFORMITY

In both transverse and crushing tests these results show improved uniformity due to the control methods. Not only are the strengths of the specimens from one project more uniform, but the various projects are more uniform with each other.

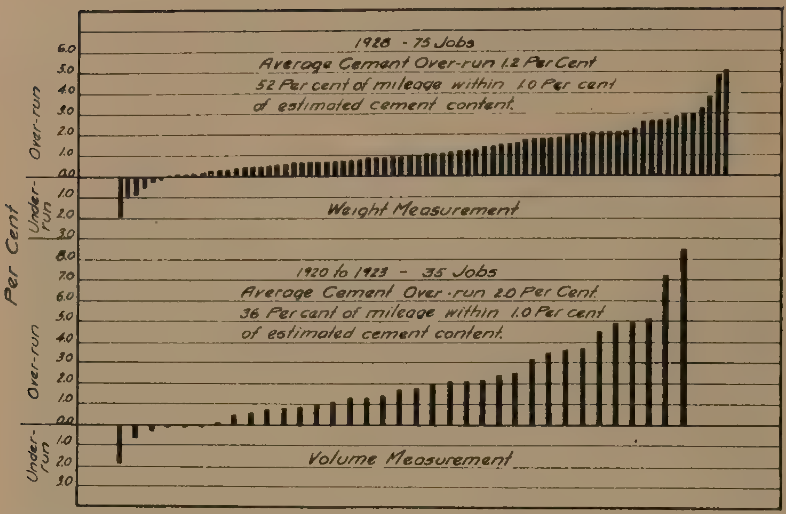

Figure 14. The Length of Each Line Shows the Percentage of Overrun or Underrun in Cement Used on Each Job

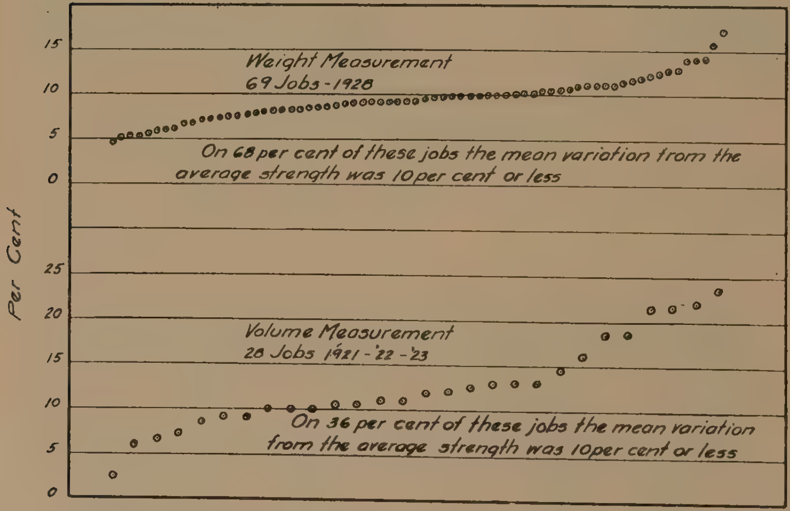

Figure 15. Each Point Represents the Mean Variation from the Average Strength of the Compression Tests on One Project, Showing That the Controlled Projects of 1928 Had in General More Uniform Strength

Table IV shows for each project in 1928 the mean variation of the strength of the individual specimens from the average of all of the specimens for the project. In the transverse tests the least uniform project had a mean variation of 17.8 per cent and the most uniform 4.6 per cent. In compression the least uniform was 17.2 per cent and the most uniform 4.7 per cent. On 28 projects under volumetric measurement in 1921-22 and 1923, the least uniform project had a mean strength variation from the average of 23.5 per cent and the most uniform was 2.5 per cent. In 1928 the mean variation of 68 per cent of the jobs was 10 per cent or less. In 1921 to 1923 only 36 per cent had a mean variation of 10 per cent or less.

Figure 15 shows these data graphically. As the mean variations for each job are plotted in the order of magnitude, the average slopes of the resulting lines offer a ready comparison between the uniformity of the two sets of data. The less steep slope for the controlled projects indicating that the jobs were more uniform with each other.

Figure 16 shows the range in the compressive strength of the specimens from three of the 1928 projects, having mean variations of 15.6, 10.0, and 5.6 per cent respectively. This diagram illustrates principally the fact that the mean variation from the average is a good measure of relative uniformity.

In Figure 17 the average crushing strengths of 69 jobs under control in 1928 are compared with 28 volumetric measurement jobs in 1921 to 1923. The greater uniformity of the 1928 work is again apparent.

These projects were scattered over the entire state of Iowa, and there was a considerable diversity in both fine and coarse aggregates. The fine aggregates ranged from fine river sands to coarse glacial deposits, and the coarse aggregates included several grades of gravel and soft, medium and hard limestone. In view of the degree of uniformity secured it is reasonable to conclude that this system of proportioning based upon the maintaining of constant absolute volumes of aggregates per batch, is a sound method.

EFFECT OF PROPORTIONS

Under the specifications in force the contractors had the option of using anyone of six different base proportions. The determining factor usually being the relative economy. These mixtures are expected to yield concrete of equivalent strengths when made of similar ma-

TABLE IV

Test Data—Paving Projects, Iowa Highway Commission, 1928

Project number	Coarse aggregate	Proportion weight	Water gallons per bag of cement	Per cent of est. amt. of cement used	Miles laid in 1928	Beam tests, age 10 days			Compression tests, age 27 to 30 days		
						Number of specimens	Mean variation from average, per cent	Modulus of rupture, lbs. per sq. inch	Number of specimens	Mean variation from average, per cent	Strength lbs. per sq. inch
1	Stone	1 : 1.93 : 2.91	5.42	101.25	18.48	74	10.3	592	62	14.0	3880
2	Gravel	1 : 1.93 : 2.91	4.78	100.72	6.60	58	17.8	608	67	9.6	4420
3	Gravel	1 : 2.06 : 2.53	4.54	100.95	10.35	36	12.4	454	33	8.6	4719
3A	Gravel	1 : 1.93 : 2.91	4.68	100.91	5.12	26	16.3	497	28	9.7	4962
4	Gravel	1 : 2.23 : 2.23	4.90	99.92	12.34	72	14.7	562	66	13.8	4459
5	Gravel	1 : 2.17 : 2.17	4.57	100.71	3.01	6	10.5	517	16	8.5	4962
5A	Gravel	1 : 2.17 : 2.17	4.42	100.76	15.44	103	9.8	587	98	12.6	4545
7	Stone	1 : 1.93 : 2.91	4.78	100.89	10.50	60	6.7	660	51	8.5	4943
11	Sand–Gr.	1 : 3.5	3.94	100.34	5.25	47	10.8	703	22	4.7	5177
12	Sand–Gr.	1 : 4.0	4.48	99.08	8.09	48	9.3	711	44	5.6	5190
14	Stone	1 : 1.93 : 2.88	4.95	101.03	13.90	118	12.2	492	69	11.0	3600
14A	Stone	1 : 1.93 : 2.88	5.08	100.62	5.78	18	14.4	492	31	9.5	4195
15	Sand–Gr.	1 : 4.0	4.23	100.27	15.00	86	8.8	581	62	12.3	4747
15A	Sand–Gr.	1 : 4.0	4.38	100.46	8.56	94	9.4	567	78	11.8	4535
16	Sand–Gr.	1 : 4.0	4.30	102.10	13.63	97	6.8	628	98	11.0	4541
16A	Sand–Gr.	1 : 4.0	4.63	99.72	7.61	3	7.4	668	25	10.2	4472
17	Gravel	1 : 1.93 : 2.91	4.42	100.19	12.15	57	8.0	647	47	6.2	5284
18	Stone	1 : 1.93 : 2.91	4.92	101.01	10.45	73	11.3	522	73	11.0	4345
19	Stone	1 : 1.93 : 2.86	5.12	100.71	13.94	60	7.2	639	57	9.1	4853

TABLE IV—CONTINUED

Project number	Coarse aggregate	Proportion weight	Water gallons per bag of cement	Per cent of est. amt. of cement used	Miles laid in 1928	Beam tests, age 10 days			Compression tests, age 27 to 30 days		
						Number of specimens	Mean variation from average, per cent	Modulus of rupture, lbs. per sq. inch	Number of specimens	Mean variation from average, per cent	Strength lbs. per sq. inch
20	Various	1 : 2.06 : 2.53	4.62	102.04	16.39	23	14.8	450	61	10.5	4376
21	Gravel	1 : 2.06 : 2.53	4.62	102.74	5.95	62	9.6	579	62	12.2	4213
21A	Gravel	1 : 2.06 : 2.53	4.23	100.79	2.54	24	13.1	521	11	8.2	4595
23	Gravel	1 : 2.02 : 3.09	3.78	102.65	14.83	59	11.6	582	46	10.0	4594
24	Stone	1 : 2.17 : 2.17	4.40	100.68	11.60	48	13.0	555	50	13.9	4168
25	Stone	1 : 2.17 : 2.17	5.08	102.01	8.85	51	10.8	605	48	7.3	4919
26A	Gravel	1 : 2.02 : 3.09	4.49	102.64	17.81	82	7.8	635	94	9.7	4593
27	Stone	1 : 1.93 : 2.80	5.25	101.00	14.55	58	11.7	509	46	12.6	4397
28	Gravel	1 : 1.93 : 2.91	4.58	100.32	7.59	45	9.7	579	35	9.6	4714
31	Gravel	1 : 1.76 : 3.58	4.10	100.94	9.31	43	6.6	651	44	10.9	4782
32	Gravel	Various	4.04	101.75	8.92	81	6.8	678	58	8.7	4727
33A	Gravel	1 : 2.01 : 3.03	3.97	103.85	6.49	15	10.0	714	14	5.3	5404
35A	Gravel	1 : 1.97 : 2.97	4.38	105.06	6.27	61	6.3	634	42	7.6	5075
38	Stone	1 : 1.97 : 2.97	4.60	100.46	10.13	41	8.2	606	46	9.1	4684
39	Stone	1 : 1.93 : 2.91	4.74	102.04	14.44	86	9.5	629	89	8.3	4652
39A	Stone	1 : 1.97 : 2.97	4.17	101.06	1.89	21	11.0	580	20	9.7	4318
40	Stone	1 : 1.71 : 3.50	4.60	102.14	5.26	26	12.0	664	24	9.1	5048
44	Stone	1 : 1.71 : 3.47	5.15	101.83	8.08	34	13.8	527	52	15.6	3776
46	Stone	1 : 1.93 : 2.91	4.70	102.17	8.19	87	11.5	556	42	6.7	3883
47	Stone	1 : 2.06 : 2.53	4.75	103.25	17.61	102	9.5	661	82	7.3	5058

TABLE IV—Continued

Project number	Coarse aggregate	Proportion weight	Water gallons per bag of cement	Per cent of est. amt. of cement used	Miles laid in 1928	Beam tests, age 10 days			Compression tests, age 27 to 30 days		
						Number of specimens	Mean variation from average, per cent	Modulus of rupture, lbs. per sq. inch	Number of specimens	Mean variation from average, per cent	Strength lbs. per sq. inch
47A	Stone	1: 2.06: 2.53	5.02	104.88	7.33	76	12.2	605	50	9.7	4579
49	Stone	1: 1.71: 3.52	4.67	101.56	16.84	64	8.3	695	99	10.4	4707
50	Stone	1: 2.11: 2.63	4.15	100.04	10.59	51	6.6	679	41	8.4	4877
50A	Stone	1: 2.11: 2.63	4.18	100.06	5.08	28	9.3	630	13	5.9	4467
52	Stone	1: 2.11: 2.63	4.49	98.03	6.62	40	7.8	538	38	9.1	4450
54	Stone	1: 1.96: 2.96	5.03	100.00	18.59	133	7.2	570	98	9.2	4288
56	Stone	1: 1.96: 2.91	5.00	100.13	22.26	104	5.8	*576	72	9.1	3732
57	Gravel	1: 1.93: 2.95	5.02	102.11	7.02	70	8.7	514	55	11.6	4252
59	Gravel	1: 2.06: 2.53	4.90	98.99	4.44	12	4.6	642	20	8.1	4481
60	Gravel	1: 2.06: 2.53	4.95	100.48	19.47	71	11.0	601	75	9.9	4263
62	Gravel	Various	4.93	101.81	13.37	114	7.7	630	75	9.3	4625
63	Gravel	1: 1.93: 2.91	4.82	100.74	8.68	33	5.5	614	41	9.9	4462
64	Sand–Gr.	1: 4.0	4.72	100.41	5.27	45	9.2	634	28	10.3	4071
65	Stone	1: 1.93: 2.91	5.27	101.56	5.01	52	8.5	586	54	11.0	4163
66	Gravel	1: 1.83: 3.61	4.16	102.30	1.39	21	8.3	725	12	5.3	4975
67	Stone–Gr.	1: 1.93: 2.91	5.22	100.66	13.15	99	10.0	589	114	9.1	4144
68	Gravel	Various	4.20	101.39	7.90	78	10.0	528	64	7.9	4321
70	Stone–Gr.	1: 1.93: 2.91	4.83	101.55	5.53	38	7.9	682	11	6.8	4950
71	Stone	1: 1.93: 2.91	4.86	101.11	9.86	83	9.6	655	52	7.4	5105

* 7 day test.

TABLE IV—CONTINUED

Project number	Coarse aggregate	Proportion weight	Water gallons per bag of cement	Percent of est. amt. of cement used	Miles laid in 1928	Beam tests, age 10 days			Compression tests, age 27 to 30 days		
						Number of specimens	Mean variation from average, per cent	Modulus of rupture, lbs. per sq. inch	Number of specimens	Mean variation from average, per cent	Strength lbs. per sq. inch
72	Stone	1: 1.93: 2.91	4.60	101.82	8.50	57	8.4	654	42	8.2	4541
72A	Stone	1: 1.92: 2.83	4.68	100.68	6.52	40	7.5	633	16	6.0	4979
73	Gravel	1: 1.93: 2.91	4.77	99.44	5.34	32	8.2	599	37	7.7	4104
74	Gravel	1: 2.17: 2.17	4.82	102.14	4.78	70	7.3	631	41	9.0	4545
75	Gravel	1: 1.93: 2.94	4.72	102.93	3.33	36	7.3	617	8	5.1	5079
76	Gravel	1: 1.93: 2.91	4.67	101.45	8.00	36	16.4	575	26	17.2	4046
77	Gravel	1: 2.06: 2.53	4.77	100.51	2.90	28	7.8	549	25	10.1	4061
78	Gravel	1: 1.93: 2.91	4.40	100.91	3.59	22	6.8	572	35	7.5	4055
78A	Gravel	1: 1.93: 2.91	4.47	101.20	3.02	14	6.8	623	26	10.7	4267
81	Gravel	1: 2.06: 2.53	4.76	101.22	4.69	16	12.9	641	25	11.4	4314
81A	Gravel	1: 1.93: 2.91	4.88	101.27	2.26	6	7.0	604	17	10.4	4434
	Total				634.23	3754			3303		
	Average			101.20				600			4538

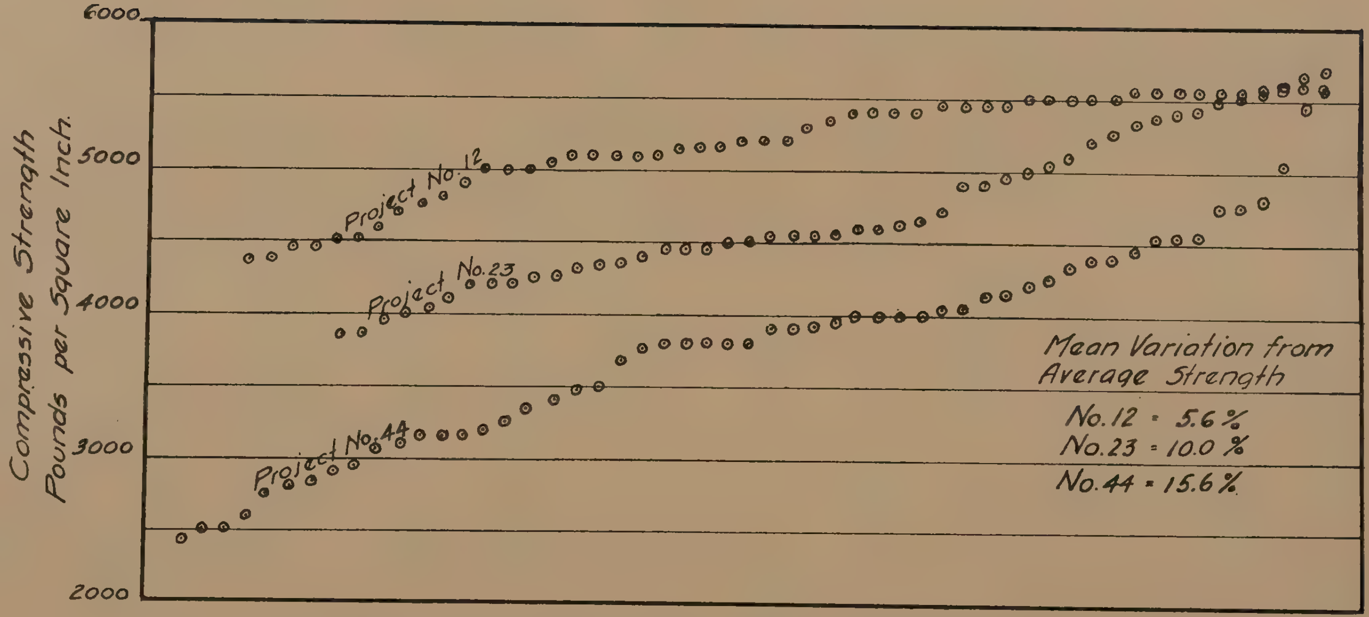

Figure 16. Compressive Tests from Three Projects of Low, Average and High Mean Variation Arranged in the Order of Magnitude. The Slope of These Is Also an Indication of Comparative Uniformity

terials and the concretes are of the same consistency. In order to produce the same consistency for various mixtures, it has been found necessary to have a practically constant water-cement ratio.

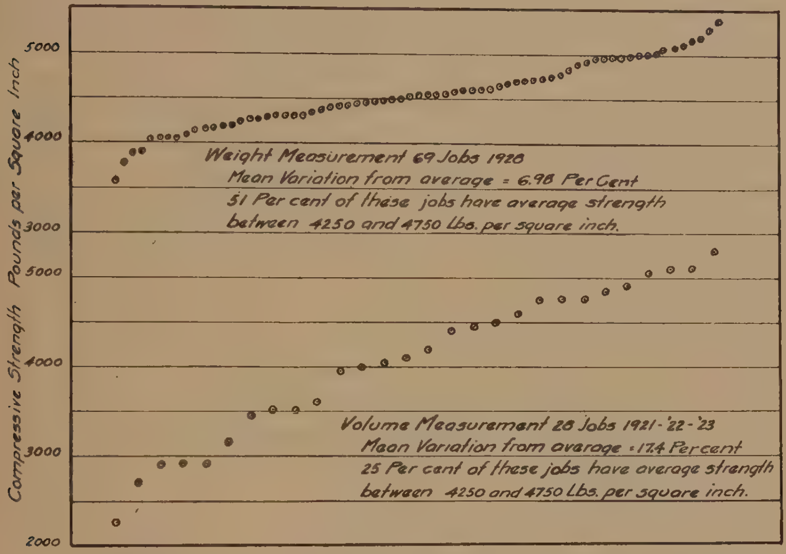

Figure 17. Average Compressive Strengths. Each Point Represents the Average Strength of the Specimens Made Each Day on One Job

TABLE V

Effect of Various Base Proportions Upon Strength—Iowa 1928

Base proportions by weight	Number of projects	Average strength tests					
		Transverse modulus of rupture, lbs. per sq. inch			Crushing, lbs. per sq. inch		
		Maximum	Minimum	Average	Maximum	Minimum	Average
1—1.71—3.47	5	725	527	652	5048	3776	4658
1—1.93—2.91	35	714	492	595	5404	3600	4497
1—2.06—2.53	13	679	450	580	5058	4061	4496
1—2.17—2.17	6	631	517	576	4962	4168	4598

In 1928 four of the permissable proportions were used. Table V showing the average transverse and crushing strengths of the specimens from all of the projects demonstrates that the same class of concrete was produced with each of the mixtures.

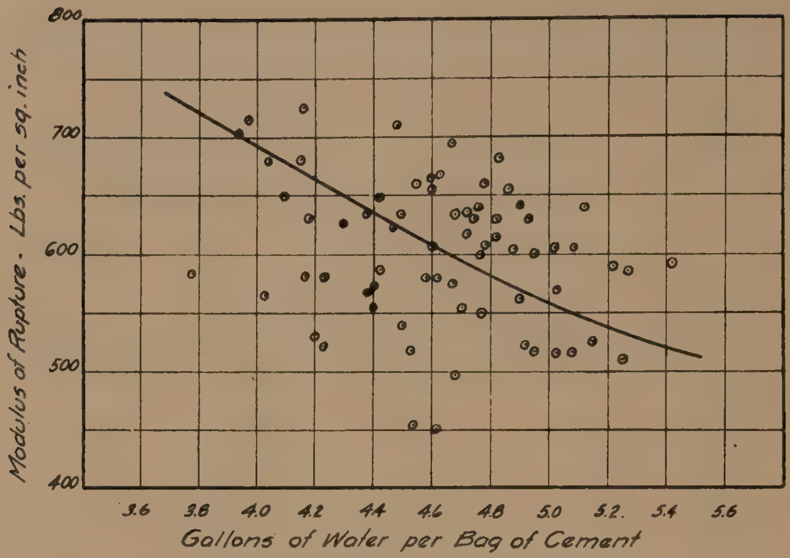

Figure 18. Relation between Average Transverse Strength and Average Water-Cement Ratio. Each Point Is the Average for One Job

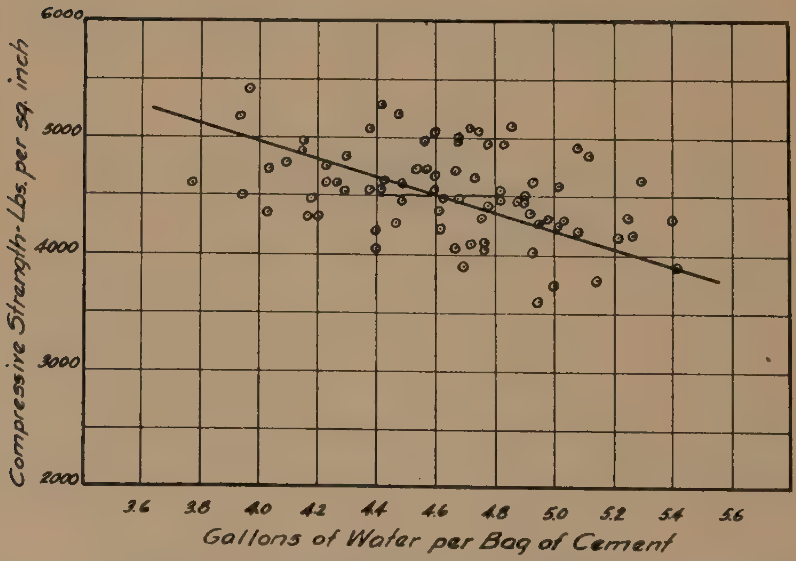

Figure 19. Relation between Average Compressive Strength and Average Water-Cement Ratio. Each Point Is the Average for One Job

The record of amount of water used was, as previously mentioned, rather unreliable, however, when the average job strengths are plotted against the water-cement ratio as in Figures 18 and 19, the general concordance of these data with those of all other experiments is noted.

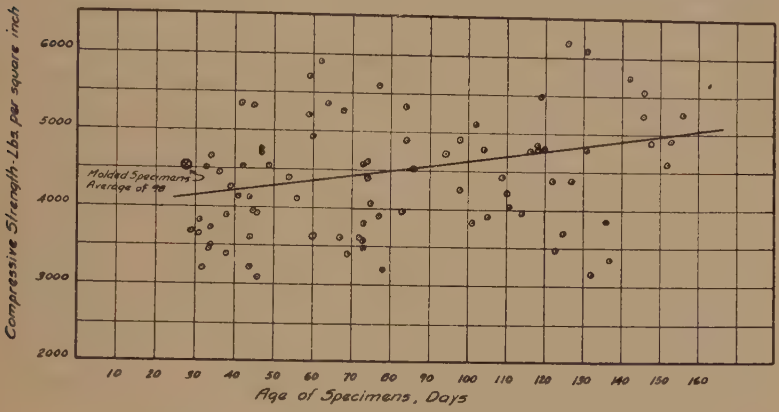

Figure 20. Tests of Individual Cores. Project No. 16

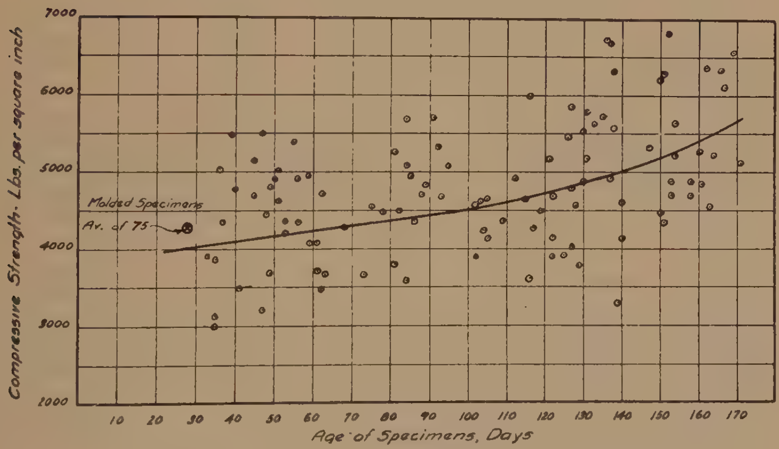

Figure 21. Tests of Individual Cores. Project No. 60

CRUSHING STRENGTHS FROM MOLDED SPECIMENS AND DRILLED CORES

On many of these projects cores have been taken and tested at various ages. Figures 20 and 21 showing the strengths of the individual core specimens on two jobs are typical of all. In making these tests insufficient attention was paid to the moisture content of the specimens at the time of tests. This would account for some of the strength variations. However, the data are extensive enough so that the average strength line shown on the diagrams probably represents the increase in strength of the concrete. It is interesting to note the average 28 day strength of the molded specimens is slightly higher than the indicated average core strength at the same age.

In Figure 22 data from all of the projects are shown. Each point being the average of a number of cores of approximately the same age. The molded specimen average shown is the grand average for all projects. The same relation between cores and molded specimens appears here as on the diagrams for the individual jobs.

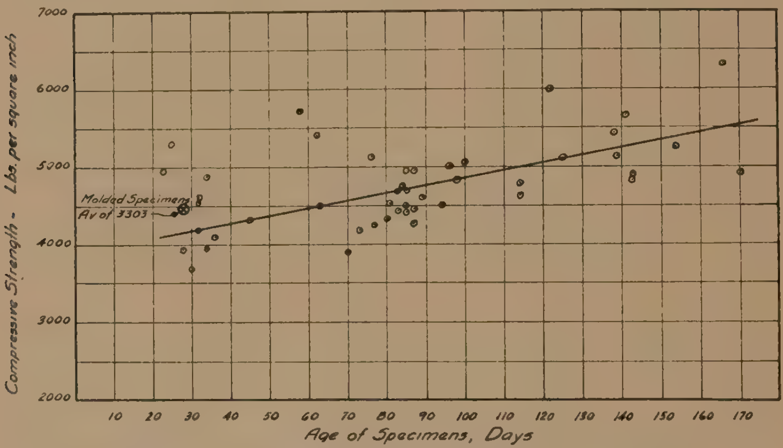

Figure 22. Tests of Cores. Each Point Is the Average of a Group of Cores of Approximately the Same Age, from One Job

KIND OF AGGREGATE

On 30 projects the coarse aggregates were of gravel and on 26 projects they were of crushed limestone. The specifications assume, that in so far as controlled by the proportions, if the mixtures contain the same absolute volume of aggregates and cement the strengths will be the same. This assumption is substantiated by these data, the average strengths of gravel and limestone concretes being practically identical. Each material of course, exhibits a range in strength due to other factors that apparently effect each in about the same degree. Figure 23 shows the average strengths of the various jobs separated into gravel and limestone.

The Iowa practice of proportioning both types of materials upon exactly the same basis has been successful in yielding concrete of the quality expected.

The range in physical characteristics of the aggregates is shown in Table VI.

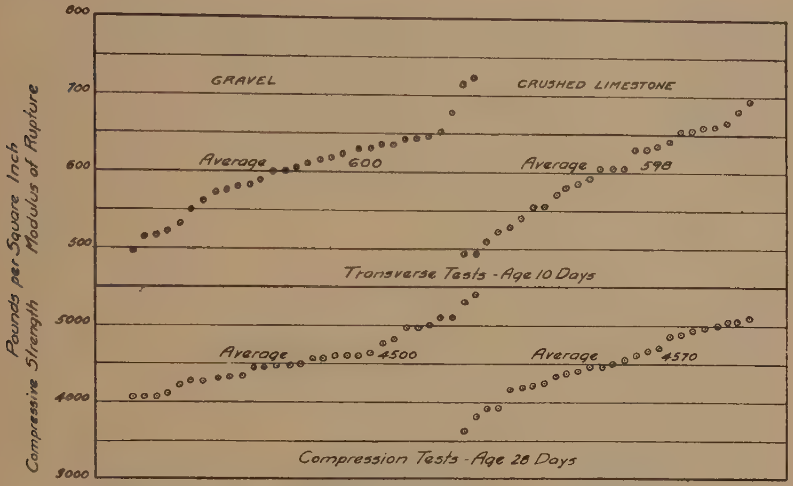

Figure 23. Gravel and Crushed Limestone Aggregates. Each Point Is the Aggregate for One Project

TABLE VI

CHARACTERISTICS OF IOWA AGGREGATES

	Range	
Sieve analysis	Gravel, per cent	Limestone, per cent
Passing 2½-inch sieve	100	100
Passing 1½-inch sieve	85 to 100	80 to 100
Passing ¾-inch sieve	44 to 74	30 to 70
Passing ⅜-inch sieve	8 to 34	10 to 21
Passing No. 4 sieve	1 to 4	1.5 to 5
Silt and Clay	0 to trace	... to ...
Shale	0 to 0.5	... to ...
Coal	0 to 0.25	... to ...
Sticks	0 to trace	... to ...
Clay Lumps	0 to trace	... to ...
Soft Particles	0 to 10.3	0.6 to 10.3
Percentage of Wear	*3.6 to 30.0	†4 to 8
Specific Gravity	2.62 to 2.70	2.53 to 2.70

* Modified Abrasion Test for Gravel—A. A. S. H. O.
† Deval Abrasion Test—A. S. T. M.

The gravels of Iowa are for the most part of glacial origin and contain from 0 per cent to 50 per cent of limestone particles.

RELATION BETWEEN TRANSVERSE AND CRUSHING STRENGTH

The individual specimens exhibit about the same relative range in strengths in bending as in compression, as do also the average for the

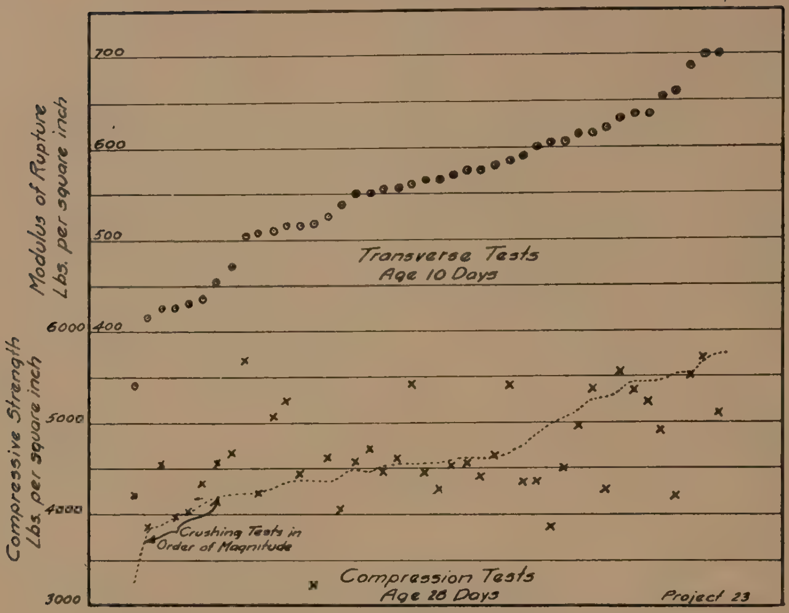

Figure 24. Relation between Transverse and Compressive Strengths of the Same Specimen. Transverse Tests Platted in the Order of Magnitude, with the Crushing Strength of the End of the Same Beam Directly Below

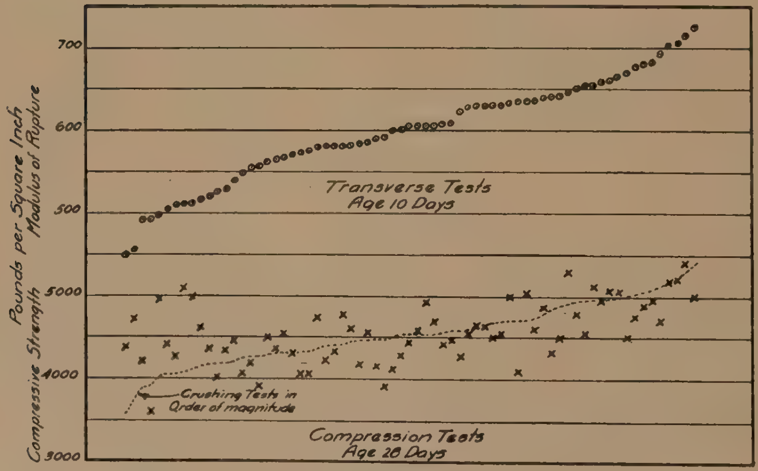

Figure 25. Relation between Transverse and Compressive Strength, Job Averages. Average Transverse Strengths of Jobs—Platted in the Order of Magnitude, with the Corresponding Average Crushing Strengths Directly Below

various jobs. The same basis for design of mixtures therefore appears to apply to either case.

These data do not however, exhibit any constant relation between the transverse and compression strength of the individual specimens, or the averages of individual jobs. In Figure 24 the modulus of rupture of the individual specimens are plotted in the order of magnitude. Directly beneath each transverse test is shown the crushing test on the same specimen. In Figure 25 the average strengths of the specimens from all of the projects are compared in the same way. The general relation indicated by these data between transverse and crushing strength, show that in so far as mixtures are concerned, the same general laws apply, but that in individual cases the transverse and compressive strengths are differently affected by other conditions and characteristics of the materials. For instance it is well known that the effect of the moisture contained in the specimen is very different for the two kinds of stress.

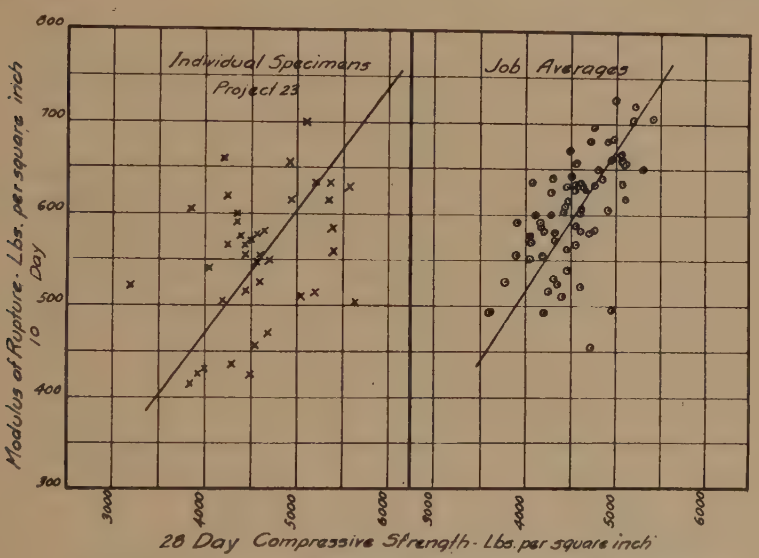

Figure 26. Relation between Transverse and Compressive Strength

General Conclusions

The pavements built under these methods of control cost no more than under the older systems. In fact it is believed that the definiteness of the quantities of materials required by the specifications has been a factor in lowering costs.

The cost of inspection has not been increased although higher grade inspectors are needed than formerly.

The concrete is of high quality and of more uniform character than formerly.

It is therefore reasonable to conclude that the results of the methods of control of the quality of concrete described are more than commensurate with the efforts expended.

DISCUSSION

ON

CONTROL OF MATERIALS AND MIXTURES FOR PAVEMENTS

Bert Myers, *Engineer of Materials and Tests, Iowa Highway Commission.* In the paper just presented Mr. Crum has described the application of a specification that contains most of the features that seem to be essential and desirable in a specification for concrete.

1. It provides for the use of the materials available locally in a manner consistent with economy and adequate strength.
2. It avoids specifying both method and result.
3. It insures adequate minimum strength.
4. It provides uniform quality of concrete from various proportions of a single kind of cement, a single kind of sand and a single kind of coarse aggregate.
5. It provides a basis for a reasonably accurate estimate of the cost of the concrete.

The data presented show that accurate control of the quality of concrete on any given job has been accomplished in a degree that seems quite satisfactory. These data do not indicate that the control of the quality of the concrete over a broader field which includes the use of various kinds of cement, sand and coarse aggregate is as accurate as might be desired. This specification does not attempt to provide for a special design of concrete mixtures for each combination of different kinds of materials. While the desirability of such a provision has been recognized by the writers of this specification, a satisfactory method of incorporating such a provision has not been apparent to them.

In the selection of the design for a concrete mixture, the engineer is faced with two considerations that are involved in any problem of engineering design. These are quality and cost. A specification for concrete is not adequate unless it insures adequate and uniform quality and provides a basis for an accurate estimate of cost.

The specification under discussion provides for concrete of adequate minimum strength. It provides a basis for an accurate estimate of the cost of the cement which is the most expensive item of material in the concrete.

This specification does not provide for concrete of the same strength on two jobs unless the kind of cement, sand and coarse aggregate is the same for both. Mr. Crum reports a variation in average compressive strength at 28 days from 3600 to 5404 pounds per square inch for various jobs. This is a variation of 50 per cent of the minimum or a variation of approximately 21 per cent each way from the average for all the jobs reported.

This specification does not provide a basis for an absolutely accurate estimate of the quantity of aggregates required for a given volume of concrete. The following example represents an extreme case.

The base proportion specified was 1-1.946-2.919 by weight. The estimated quantity of cement was 1.71 barrels per cubic yard of concrete. This estimate was based on the assumption that the water-cement ratio would be 5.25 gallons per bag. It was found on the job that the water cement ratio required to produce concrete of the desired workability with the materials actually used was 4.27 gallons per bag. This resulted in a yield of concrete per barrel of cement 3.01 per cent less than it would have been had the water-cement ratio used been that which was assumed in arriving at the estimated quantity of cement. Had the base proportion been used with a water-cement ratio of 4.27 gallons per bag the pavement would have cost the contractor approximately 3 per cent more than might have been anticipated. This is true because the yield per batch would have been decreased, thus reducing the output of the crew per unit of time. The only items of expense not increased by this deficiency in yield would be the earth work, perhaps the form setting and perhaps the curing.

The proportions on this job were adjusted to add a sufficient quantity of aggregates to produce the anticipated yield using the estimated quantity of cement per cubic yard. This change in proportions resulted in the use of 4.17 per cent more aggregates than was anticipated. If the sand cost $1.10 per ton and the coarse aggregate cost $1.90 per ton delivered, the cost of the pavement was increased 1.52 per cent. Thus the discrepancy between the estimated cost based on the data given in the specifications and the actual cost was reduced approximately one-half by the adjustment of proportions. Since the change in the water-cement ratio necessary to maintain the desired workability with this slight increase in the quantity of aggregate was negligible, the owner still secured concrete of a quality well above that which would have resulted had the base proportions been used with a water-cement ratio of 5.25 gallons per bag. The average compressive strength at 28 days of the concrete on this job was 4220 pounds per square inch.

Improvement in this specification both as to uniformity of strength of concrete for all possible combinations of different kinds of materials and as to providing a basis for a more accurate estimate of cost would be desirable. Improvement in one of these at the expense of the other would not be desirable.

The science of the design of concrete mixtures does not seem to have been developed to such a point as to provide a general solution of the problem thus presented.

By applying any one of several theories of design of concrete mixtures, the engineer may arrive at a specification for concrete mixtures which may be expected to provide for concrete of any required strength. However, none of these theories provides any convenient means for arriving at a basis for accurately estimating the quantity of the various materials necessary to produce a given quantity of concrete. For the solution of this problem of yield of concrete the method of trial batches at once suggests itself. This would seem a practical method of solution if the number of possible combinations of different kinds of cement, sand and coarse aggregate were small, and provided some dependable measure of workability were available.

Mr. Crum has stated that there is no measure of workability that can be depended upon to give results that are reliable for field use. Assuming, however, that it is possible to make up, in the laboratory, trial batches of concrete which can be depended upon to prove workable in the field, on account of the number of variables encountered the problem is still not simple when applied to highway work covering an entire state.

During 1929 there were used on highway work in Iowa, cement from twenty-three different mills, sand from forty-two different sources and coarse aggregate from forty-two different sources. This gives 40,572 possible combinations of cement, sand and coarse aggregate.

Supposing that the Iowa Highway Commission should desire to determine the yield for each of these combinations for each of the six specified proportions and that the solution in each case required the mixing of ten trial batches. This would mean the mixing of 2,434,320 batches of concrete which, at the rate of ten batches per hour, would require the work of one crew for a little over twelve years. Thus the amount of work necessary to give all the data necessary to estimate accurately the quantity of all materials necessary to produce a cubic yard of concrete from each of the possible combinations is quite imposing, even if it is assumed that any kind of material from a given source is always the same as regards the characteristics which affect yield of concrete.

In actual practice the number of variables entering into the solution of the problem is, of course, reduced materially since the materials from all sources would not be available for use on any given job.

One case in which this problem was greatly simplified was that described by R. W. Crum, Proceedings of the Seventh Annual Meeting of the Highway Research Board, Part I, page 212. In this paper is reported the procedure followed in designing a mixture for a single aggregate which was the only aggregate available within a considerable area.

The mixture was designed after methods described by Talbot and Richart, Bulletin No. 137, Engineering Experiment Station, University of Illinois. In the paper mentioned above Mr. Crum reported the preliminary work of design.

From a further study based on more complete data, the following design was obtained. The strength required was 3500 pounds per square inch at 28 days. The laboratory studies indicated that the relative water content necessary to produce concrete of satisfactory workability was 1.10. According to Talbot and Richart, concrete with basic water content would be 1.06 times stronger than this concrete. Therefore the strength at basic water content would need to be 1.06 × 3500 or 3710 pounds per square inch. Referring to the void strength curve for this material, it was found that the void-cement ratio corresponding to this strength is .85 cubic feet per bag. Referring to the characteristic curves for 1.1 relative water content, we find that for .85 cubic feet of voids per bag the ratio of aggregate to cement by absolute volume would need to be 4.6 and that the unit composition of the concrete would be as follows:

Cement	.136 cu. ft. abs. vol. per cu. ft. of concrete.
Aggregate	.626 cu. ft. abs. vol. per cu. ft. of concrete.
Water	.181 cu. ft. abs. vol. per cu. ft. of concrete.
Air voids	.057 cu. ft. abs. vol. per cu. ft. of concrete.
	1.000

Reducing the composition to pounds, we find the composition to be as follows:

Cement $= .136 \times 195.4 = 26.58$ lb. per cu. ft. of concrete.
Aggregate $= .626 \times 163.4 = 102.30$ lb. per cu. ft. of concrete.
Water $= .181 \times 62.5 = 11.31$ lb. per cu. ft. of concrete.

The mix is, therefore, 26.58 lbs. cement to 102.30 lbs. aggregate = 1 to 3.85.

The water cement ratio is 11.31 lbs. per 26.58 lbs. cement = $\frac{11.31}{26.58} \times 94 = 40.0$ lbs. per bag of cement = 4.80 gals.

The concrete will require 26.58 lbs. cement per cu. ft. = 27 × 26.58 lbs. = 717.66 lbs. = 1.91 bbls.

TABLE I

Comparison between Design Data and Construction Data for Concrete Mixture

	Water, gal. per bag	Cement, bbl. per cu. yd.	Aggregate cement ratio by weight	Air voids, cu. ft. per bag	Void cement ratio, cu. ft. per bag	Strength, lb. per sq. in. at 28 days
Design data	4.80	1.91	3.85	.2018	.842	3500
Construction data	4.62	1.92	3.96	.1802	.796	4310
Specified	...	1.90	4.00	....	...	....

Previous experience indicated that the quantity of water that would be used in the field was likely to be somewhat less than that assumed in the design. Trial batches of concrete of approximately the same design indicated that the strength was likely to be somewhat higher than that for which the concrete was designed. From this information the specification for this mixture was fixed at a proportion of 1 to 4.00 by weight with an estimated quantity of cement of 1.9 barrels per cubic yard of concrete.

The data from eight jobs using this mixture on approximately 85 miles of pavement constructed in 1929 provide an interesting check on the accuracy of this design. Table I shows a comparison between data used in the design of the mixture and the average of data obtained in the field from the eight jobs mentioned above.

As was anticipated, certain slight discrepancies between the laboratory and field data are apparent. The amount of mixing water used in the field was less than that assumed in the design. In other words, the desired workability was obtained in the field with less mixing water than the quantity indicated as necessary by laboratory studies. This is believed to be due to more thorough mixing obtained in a paving mixer than is obtained by hand mixing in the laboratory. The design data were obtained from mortar specimens mixed by hand according to usual laboratory practice.

The same assumption offers a satisfactory explanation of the reduction in the quantity of air voids.

The reduction in the void cement ratio accounts for some of the difference in strength between that used in design and that obtained in the field. Reference to the void strength curve for this material shows that concrete with a void cement ratio of .796 cubic foot per bag of cement and with the relative water content might be expected to have a compressive strength of 3900 pounds per square inch at 28 days. The remaining difference of 410 pounds per square inch may be explained by the fact that the laboratory specimens were cured at a temperature of 70 degrees Fahrenheit while the specimens from which the field data were obtained were cured at natural summer temperatures in the field.

The increase in the quantity of dry materials necessary to produce a cubic yard of concrete results naturally from the decrease in the void cement-ratio. It would appear, therefore, that had there been available some means of determining accurately in the laboratory what mixture would prove to have proper workability in the field, a mixture would have been designed that would have been accurate both as to strength and the quantity of all materials necessary to produce a given volume of concrete.

The degree of success attained in solving a special problem in the design of concrete mixtures indicated by the example cited has the tendency to inspire the hope that it is possible to arrive at a general solution for all problems in the design of concrete.

It would not seem impossible to arrive at some measure of workability for concrete to be used in pavements. It would also seem that there must be some characteristic of any given material which affects yield when that material is used in concrete. If so, it would seem to be within the realm of possibility to discover this characteristic and measure it. Some engineers have suggested the term "water demand" as descriptive of this characteristic. An example of this which is commonly noted is the amount of water required to produce normal consistency in cement paste. This quantity for one kind of cement may be 23 per cent of the quantity of cement and for another 26 per cent. The amount of water necessary to produce concrete of any given degree of workability will not be the same when one of these cements is used as it will be for the other. Hence the yield of concrete for a given quantity of dry materials will probably not be the same for both.

Doctor Talbot has reported the difference in the void content of mortars of the same consistency mixed with the same cement and different sands.

There seems to be some indication that the same phenomenon is encountered in coarse aggregates from different sources. If this characteristic could be measured independently for each kind of material, it would then seem possible to estimate the yield of concrete from any combination of kinds of materials. So far this has not been done.

This would seem to present an interesting field for much needed research. It is to be hoped that such research will be conducted and that the results will provide some rational method for a more general application of the recognized principles of design of concrete mixtures. There are considerable advantages to be gained by substituting accurately designed mixtures for the arbitrary proportions now commonly used if this can be done without violating the general principles which apply to the writing of specifications. The engineer cannot feel that he is performing his proper function unless he is making every practical use of the scientific knowledge available. It would seem, therefore, that there is a sufficient stimulus to insure the accomplishment of the work necessary to make the science of the design of concrete mixtures generally applicable to highway work.

F. H. Jackson, *Senior Engineer of Tests, United States Bureau of Public Roads.* The description of the methods used by the State of Iowa in controlling the quality of concrete for pavements, as given by Mr. Crum, as well as his discussion of the results which have been secured, are of peculiar interest to the Bureau of Public Roads just at this time because of the efforts which the Bureau is making to secure improvements along this line in connection with the construction of federal aid pavements. From a strictly theoretical point of view, there can be no question as to the desirability of exercising sufficient control over construction processes so as to insure that concrete of definite and uniform quality will be produced. However, there seems to be some difference of opinion among engineers as to the extent to which they are warranted in carrying their field control operations in order to secure this end, especially if such control adds to the cost of the work.

There appears to be a feeling in some quarters that, for any real improvement over existing methods of control, the additional cost would be out of all proportion to the benefits derived. The results which have been obtained in Iowa should go far towards demonstrating not only the practicability of close field control but also that significant benefits in the shape of increased uniformity are secured at practically no additional cost. For instance, the author, by comparing the uniformity of the strengths obtained on 28 jobs constructed during 1921, 1922 and 1923 with the uniformity of the strengths obtained on 69 jobs constructed in 1928, has shown that, whereas in the former case only 35 per cent of the jobs showed a mean variation from the average strength of 10 per cent or less, in the 1928 work 68 per cent of all jobs showed a variation of 10 per cent or less. It will be observed, furthermore, that the most significant change in construction practice during this interval was the abandonment of the volumetric and the adoption of the weight method of measuring aggregates. It is fair to assume that the marked increase is uniformity which has been observed is due in a large measure to this change. It has also been noted that the variation in yield has been quite materially decreased on the jobs where weight measurement was employed. As has been shown by the author, the average deviation on 75 jobs constructed in 1928 was only 1.2 per cent, which is so slight as to be entirely accounted for by variations in depth of subgrade. It is interesting to note that the theoretical principle to the effect that variations in grading of aggregates do not affect yield when measurements are made on a weight basis is so strikingly verified by actual construction practice in Iowa. This principle, to the mind of the writer, furnishes one of the outstanding advantages of weight measurement in that it makes possible a much closer estimate of quantities by the contractor, resulting in closer bidding and ultimately lower costs.

The writer would like to call attention also to the fact that the necessary field operations incident to the proper control of mixtures, such as the determination of specific gravity and moisture content of aggregates and the necessary field adjustment of mixtures, are being made at practically no additional cost for inspection, this in spite of the fact that higher grade inspectors are employed. The experience of the State of Iowa is of great interest to us because it proves that the theory of weight proportioning of aggregates can be put into practice without unduly adding to the cost of construction.

The method used by Iowa in allowing the contractor a choice of a number of base proportions, all of which are designed to give concrete of substantially the same quality, is also of interest from the economic standpoint, in that it permits the use of mixtures containing relatively high percentages of sand in those sections of the state where fine aggregate is much more abundant and consequently much cheaper than coarse aggregate. The practice of designing mixtures so as to insure the production of concrete of the desired quality at the lowest cost by making the best use of locally occurring materials is one which should be actively encouraged. We should keep in mind at all times the fact that it is the quality of the finished product which is of primary importance and that the properties of the various constituent materials are only of importance in so far as they affect the methods and cost of securing this result. It should always be remembered that real design should include consideration of the economic as well as the engineering features of the problem and that the goal should be the securing of concrete of the required quality at the lowest cost. Strict adherence to the same arbitrary proportions over an entire state where local conditions as to available materials differ widely does not properly take account of the cost feature and should be modified in favor of design methods which take local conditions into account.

It is interesting also to note that the base proportions which appear in the Iowa Specifications have each been so adjusted that concrete of substantially the same consistency will be produced with the same water-cement ratio. Subject to the inaccuracies of job water control, which are discussed by the author, the concrete strengths obtained should therfore be an indication of the soundness of the water-cement ratio strength law. The author has pointed out that the results secured indicate that concrete of the same class was produced with each of the base proportions, thus demonstrating the soundness of the design. Experience in Iowa also seems to indicate that, within the range of aggregates available in that state, there is no relation between type of aggregate and strength of the concrete either crushing or transverse. The writer would like to have available similar data from other states where the range in character of aggregates is greater than in Iowa in order to determine whether laboratory investigations carried out by the Bureau of Public Roads which indicate that type of aggregate does have an appreciable effect upon strength for a given water-cement ratio would be substantiated by field tests such as are reported in this paper. If the differences which are noted in the laboratory are not reflected under controlled field operations it is time we knew it.

If it is possible to secure sufficiently accurate results by the use of the basic water-cement ratio strength relation, the design of concrete for pavements will be greatly simplified. The writer, however, has always had a feeling that, in localities where types of aggregates vary widely, a refinement of the original water-cement ratio method of design, using the so-called "trial" method, is necessary in order to properly evaluate all of the factors affecting strength. In this connection, the question arises whether to a certain extent the dispersion of the plotted points in Figure 18 which shows the relation between water-cement ratio and modulus of rupture for the 1928 work is not at least partly due to the effect of character of individual aggregates on strength. It is noted that the band of plotted points covers a strength range of about 150 pounds per square inch, which is equivalent to the difference in strength produced by a change in the water-cement ratio of about 1 gallon per bag of cement, or 0.13. This happens to be almost exactly the value observed in the Bureau tests in which 17 different coarse aggregates were used and their effect upon strength noted.

Further Control of the Uniformity of Aggregate Gradation

The writer would like to conclude his discussion of this very interesting paper with a few remarks on the control of aggregate gradation. Irrespective of the methods used in establishing the initial proportions for a concrete job, a reasonable control over the uniformity of deliveries of materials for that job is essential if we are to avoid excess adjustments of mix to take care of variations in workability due to changes in aggregate gradation. Even when proportions are based on definite absolute volumes of materials, as in Iowa, variations in workability necessitating adjustments of mix are bound to occur from time to time under the present system of using run of stock pile coarse aggregates. Of course under conditions where it is possible to exercise rigid inspection of stockpiling, re-handling operations, etc., as is the case in Iowa, the evils incident to segregation of sizes are greatly lessened. The writer has felt for a long time that the practice of screening concrete aggregate into certain primary sizes at the producing plant and later re-combining them in certain proportions in an effort to meet a specification requirement for a well graded material is illogical. No matter how carefully the sizes are blended and no matter how carefully the material is stockpiled, segregation is bound to occur. The writer believes that the next logical step in the improvement of uniformity is the proportioning of the coarse aggregate in at least two separate sizes, such as has been done for a number of years in both North Carolina and California. It is believed that the positive control over the grading of coarse aggregate which will be secured in this manner will not only result in more uniform concrete but will also make possible the designing of mixtures carrying higher percentages of coarse aggregate than have been heretofore considered advisable. The work done by the Bureau of Public Roads during the past summer at Arlington should throw considerable light on this subject. Although the results of these tests will not be available for some time, it is possible to say at this time that in so far as workability alone is concerned it was found entirely practical to place and finish concretes in which the value $\frac{b}{b_0}$, as defined by Talbot and Richart, was raised to approximately 0.84.

In this term, b represents the absolute volume of coarse aggregate in a unit volume of concrete and b_0 the absolute volume of coarse aggregate in the same volume of coarse aggregate. The ratio is taken as a measure of the limiting quantity of coarse aggregate which may be used under a given set of conditions as regards materials, methods of placing, etc. As a matter of comparison it may be said that the values of this ratio for an average $1:2:3\frac{1}{2}$ concrete road mix is approximately 0.78.

The measure of workability used in these tests was the ability of the finishing machine on the job to handle the concrete so as to secure a satisfactory surface finish free from honeycomb in not to exceed three passages of the finishing machine. The question of relative strength and density, freedom from interior honeycomb, etc., can only be decided after the concrete has been tested, which will not be done until the coming summer.

In conclusion, the writer wishes again to voice his appreciation of the fact that the actual experiences of the State of Iowa in the construction of concrete pavements under controlled conditions have been made available through this paper for the use of other organizations interested in the same subject.

C. H. Scholer, *Consulting Materials Engineer, Kansas Highway Commission.* Mr. Crum's very excellent paper leaves but little to be added on this interesting subject, but perhaps some of the experiences in Kansas where for the past three years most of our proportioning has been done by weight, may be of interest.

Kansas has for several years past been using weight proportions for nearly all concrete pavements constructed. As in Iowa, our experience indicates a much better control of the quality of the resulting material. The high degree of accuracy shown in Mr. Crum's paper could hardly be obtained in any other way.

In estimating quantities of materials required in concrete, particularly the cement content, I would like to suggest that the use of formulas including the method of absolute volumes be used only for preliminary estimates and that for checking quantities in the field that the actual yield test be made. As pointed out by Mr. Crum, there is no reason why field methods should be any more inaccurate than the same operations in the laboratory.

For the last two years we have been determining the weight per cubic foot, using the one-half cubic foot measure, of the mixed concrete on the job and using this for the control of quantities of materials and cement content per cubic yard of concrete.

Due to a lack of suitable coarse aggregate over nearly two-thirds of the State of Kansas, and the very uniform distribution of an excellent coarse sand containing some gravel, a large proportion of the concrete we are now using contains practically no coarse aggregate.

The material used as a total aggregate will have a fineness modulus, we call it gradation factor, of about 4.00; approximately 15 per cent will be retained on a No. 4 sieve, and 10 to 15 per cent will pass the No. 28 sieve; the dry and rodded weight will be about 117 lbs. per cu. ft.

For concrete pavements we use this with proportions of 1 : 3½ by dry and rodded volumes with about five gallons of water per sack of cement, including water in the aggregate. Calculating the cement requirements of such a mix by the absolute volume basis gives an estimated value of 1.83 bbls. In actual practice it is difficult to get over 1.70 bbls. of cement into a cubic yard of such concrete. To illustrate this let me give you some typical figures on a yield test as we make it in the control of our field work. This is a very simple test where weight proportions are used and is a familiar laboratory procedure adapted to field control situations. Assume a seven bag batch the actual weight of materials in such a batch will be:

Cement, 7 bags @ 94	658 lb.
Sand gravel	2870 lb.
Water, 7 @ 0.65 @ 62.4	284 lb.
Wt. of batch	3812 lb.

The weight per cubic foot of the mixed concrete is measured in a one-half cubic foot measure, is then determined and a typical value would be 142 lbs. per cubic foot. The batch then yielded.

$$\frac{3812}{142} = 26.8 \text{ cubic feet of concrete}$$

and cement required per cubic yard are

$$\frac{1.75\ (27)}{26.8} = 1.76 \text{ bbls. per cubic yard}$$

The method is of course equally well adapted to conditions where separate aggregates are used. This method makes possible very close checks upon the cement requirements per cubic yard and we find it to be very helpful and much simpler than attempts to check up on measured lengths of pavement and uniformity of subgrading. As pointed out by Mr. Crum, a variation of 0.07 inch on a 7-inch pavement will cause a 1 per cent variation in the quantities of material used and it is the speaker's conviction that a very large percentage of the variation in cement content charged against materials in the past, and attempted to be corrected by changes in proportions have been due to subgrade variations. With weight control of proportions nearly all variation in actual cement content per unit volume of finished material can be eliminated. This is clearly shown in Mr. Crum's paper. We have found this method of checking yield very satisfying to the contractor who in nearly every instance where an overrun was occurring, would otherwise demand a readjustment of proportions.

REPORT OF COMMITTEE ON CHARACTER AND USE OF ROAD MATERIALS

H. S. Mattimore, *Chairman*

Engineer of Tests and Materials Investigations, Pennsylvania Department of Highways

The committee has no formal report on Investigations to offer this year, due to the fact that the members were unanimously of the opinion that the time spent this year would be eventually more fruitful if given over to laying out and preparing a program for future investigation.

Consideration was given to some of the suggested problems in the Coordination Committee's report, viz., " cement." At the present time this material has still many unknown qualities. The present tendency is to produce a product giving high early strength, and studies of methods by which this is being accomplished, in order to determine durability of the product in concrete, are well worthy of investigation.

Some high early strength cements have been marketed for the past few years, and a large quantity of this product is used in highway work. The present specifications are based entirely on strength tests. It does not seem as though this is sufficient to definitely control this material, therefore, it is considered advisable to form a new subcommittee for the study of Portland cement, and the high early strength cements.

The subject of workability-concrete tests that will indicate the workability in concrete pavements, is important, and although numerous investigations have been made to date, no tests suggested seems fully satisfactory. The committee will keep in contact with future investigations along this line, and report on the status of these studies periodically.

The effect of concrete admixtures to improve the workability is still an open question. The committee is of the opinion that no data presented to date indicates that any admixtures are beneficial for this purpose.

A study of the mixing process in concrete is extremely important, but there are not sufficient data available at the present time to indicate what effect this has on uniformity and quality.

In the field of bituminous materials and mixtures, we believe organized research should be carried on to determine suitable proportions and characteristics of aggregates for mixed in place bituminous surfaces. We also believe that considerable information is desirable on determination and classification of the important characteristics of bitumen for surface treatment and mixed in place surfaces. Subcommittees have been organized to study these problems on bituminous materials.

ACCURACY OF WATER MEASUREMENT ON PAVING MIXERS

F. C. Lang,
Engineer of Tests and Inspection, Minnesota Highway Department,
AND
Bert Myers,
Engineer of Materials and Tests, Iowa Highway Commission

Recent developments in the manufacture of concrete have resulted in great improvements in its quality and uniformity. Proportioning aggregates and cement by weight with corrections for the variables which affect the relationship between weight and the solid volume of the materials and for the water which they contain have contributed largely to the recent improvement. Development of the water-cement ratio theory of the design of concrete mixtures has called attention to the importance of the accurate measurement of mixing water.

The measurement of that part of the mixing water not contained in the aggregates is ordinarily done by means of equipment furnished by the mixer manufacturer and mounted on the mixing machine. The water measuring equipment furnished with the common type of paving mixer consists of a closed tank connected to a water supply pipe and to a pipe which discharges into the mixer drum. Inside the tank is an adjustable syphon, trough, pipe, or displacement cylinder designed to allow a controlled quantity of water to be drawn from the tank. One mixer is equipped with a tank having one movable head by which the total volume of the tank is varied. This tank is completely filled and completely emptied for each batch. The mechanism which controls the position of these adjustable devices is marked to indicate the quantity of water which may be expected to be delivered per batch for any given setting.

The tank is provided with one or more air relief valves which allow air to enter the tank as water is withdrawn, and allow air to escape as the tank is filled. These air valves are intended to allow all the air to escape from the tank and to prevent the escape of water.

A control valve is placed in the supply pipe and another in the discharge pipe. These two valves are ordinarily linked together so that when one valve is in the closed position the other is open. This arrangement allows the valves to be operated by a single lever and should be designed to prevent the possibility of both valves being open at the same time.

The types of tanks and control valves in common use at the time these tests were made (1927 and 1928) are shown in figures 1 and 2. The observations herein reported were made on this type of equipment. It will be noted that accuracy of measurement can be expected from this type of equipment only under the following conditions.

1. That the measuring tank is completely filled with water for each cycle of measurement. This condition can exist only when no air is entrapped in the tank.
2. That there is no opportunity for water to flow from the supply line while water is being discharged from the measuring tank. This condition can exist only if the valve arrangement is such that one valve is not opened until the other is closed.
3. That the quantity of water remaining in the tank is always the same for any given setting of the indicating device. In the horizontal type of tank with an adjustable syphon, this condition will not exist when the tank is tipped due to grade or superelevation unless the intake of the syphon is at the exact center of the tank in both directions for all settings of the indicator. If the intake of the syphon is not at the center, accuracy demands the application of an impractically cumbersom correction factor each time the tank is tipped.

Believing that the sources of error suggested above actually existed in the ordinary water measuring equipment furnished with concrete paving mixers and that these errors were serious in their magnitude, studies of the accuracy of water measuring equipment were undertaken by the Iowa Highway Commission * and the University of Minnesota.* The main sources of variation observed in these studies are, (1) variable quantities of air entrapped in the measuring system, (2) overlapping in the opening cycles of the control valves, (3) variable pressures in the pipe line and (4) the personal judgment of the mixer operator.

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* The work done by the University of Minnesota was under the direction of Mr. Swanberg. The work done in Iowa was under the direction of Mr. A. E. Stoddard.

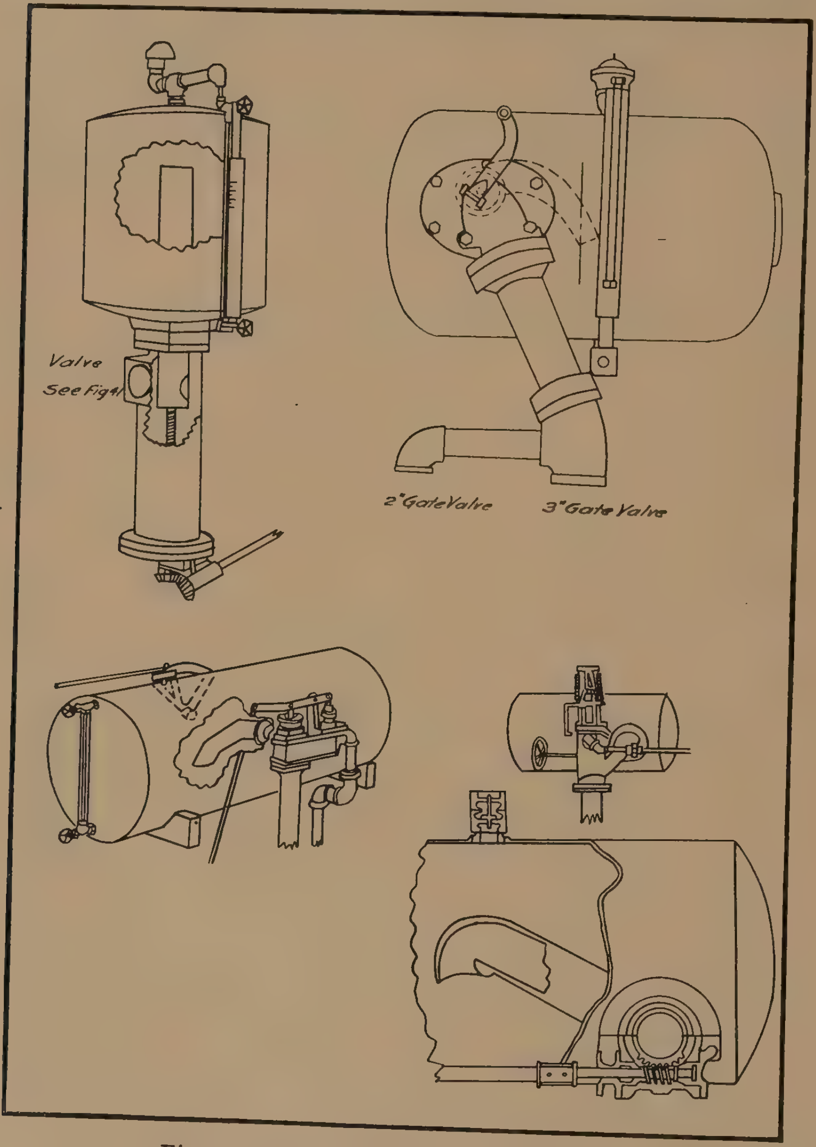

Figure 1. Measuring Tanks 27-E Pavers

There are three principal sources from which air becomes entrapped in the water supply for paving mixers. (1) Some air enters the line at the pumps due to leaky pumps and inadequately submerged intakes. (2) Some air enters the line at the joints since the joints in the pipe are not all tight and the operation of the pumps is not continuous. Air from these sources is likely to reach the mixer in small bubbles, quite uniformly distributed. (3) Each time the hose connecting the mixer with the pipe line is moved, a volume of air is admitted into the system. All this air is supposed to be eliminated by the air relief valves, but it was observed in these studies that these valves frequently fail to function completely and that they cannot be depended upon to function uniformly under varying conditions such as the quantity of air entrapped and pipe line pressure. Any air entrapped in the measuring tank displaces an unknown volume of water from the quantity required for the batch. This volume will be inversely proportional to the pressure in the tank at the time the valves are closed.

With the common design of three-way valves and connected gate valves, there was usually a period in their operation in which the intake and discharge valves of the measuring tank were both open. During this interval water flowed directly from the line into the mixer drum. If the speed of operation of the valves were absolutely uniform and the pipe line pressure always constant, the amount of water thus passing to the drum would be constant and could be taken into consideration in regulating the settings of the mechanism controlling the measuring device. In actual practice both of these factors are variable and may combine to increase or decrease this leakage.

Both of the foreging factors are affected by variations in pressure in the pipe line. With low pressure, air entrapped in the tank occupies greater space than when the pressure is high. With high pressure a greater amount of water is passed to the mixer drum in the overlapping periods of operation of intake and discharge valves.

In addition to these variables, the mixer operator frequently adds water to the batch at his pleasure by throwing the water control lever over for an additional "Squirt." No doubt this procedure is prompted by the production of "dry" batches resulting from some combination of the variables mentioned heretofore. Such action is not to be condoned because it adds another variable to an already complicated field.

The work of the University of Minnesota, which was conducted in cooperation with the Minnesota Highway Department, was a laboratory study of several types of water measuring equipment in common

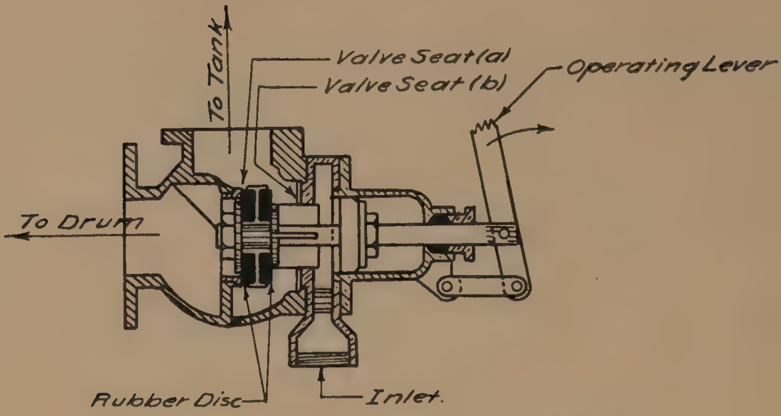
To Tank
Valve Seat (a)
Valve Seat (b)
Operating Lever
To Drum
Rubber Disc
Inlet.

Figure 2. One Type of Three-Way Valve

Note: With valve in position shown (left face of rubber disc against seat " a "), the valve is in the " fill " position, and water flows from inlet to the tank.

With valve moved to the extreme right so that right face of rubber disc is against seat " b," the inlet is closed and water flows from the tank to the drum.

If valve is at any intermediate position so that left face of rubber disc is not seated as shown, water will flow directly from inlet to drum.

This condition is typical of all valves tested.

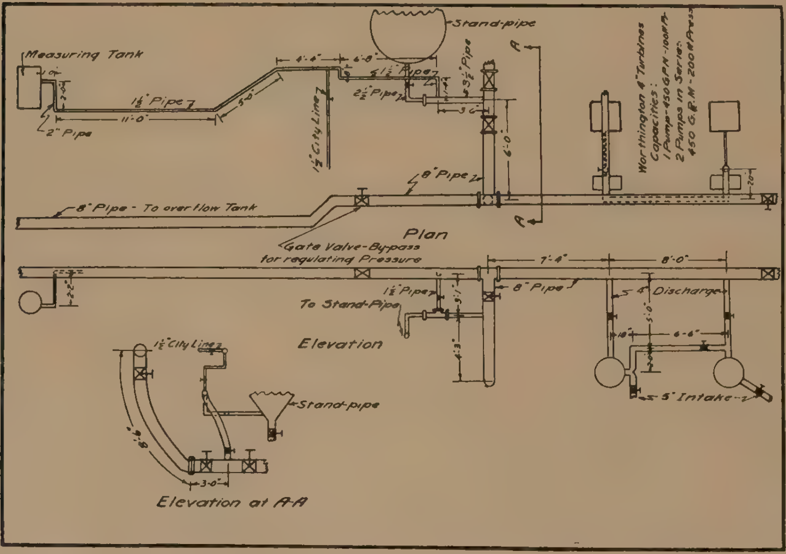
Stand-pipe
Measuring Tank
4'-4"
6'-8"
1½" Pipe
1½" Pipe
5'-0"
1½" City Line
2½" Pipe
3½" Pipe
3'-6"
11'-0"
2" Pipe
6'-0"
A
A
Worthington 4" Turbines
Capacities:
1 Pump-450 G.P.M.-100# P.
2 Pumps in Series:
450 G.P.M.-200# Press.
8" Pipe
20"
8" Pipe - To overflow Tank
Plan
Gate Valve-By-pass
for regulating Pressure
7'-4"
8'-0"
2'-2"
1½" Pipe
To Stand-Pipe
8" Pipe
4" Discharge
5'-0"
18"
6'-6"
4'-3"
Elevation
1½" City Line
Stand-pipe
5' Intake
3'-0"
Elevation at A-A

Figure 3. Arrangement of Pumps and Piping for Tests of Measuring Tanks

use. This investigation covered the effects of the following variables on four different types of tanks and one meter.

1. Accuracy of indicated quantity.
2. Effect of pressure variations on quantity of water discharged.
3. Effect of grade upon the quantity of water discharged.
4. Effect of superelevation upon the quantity of water discharged.

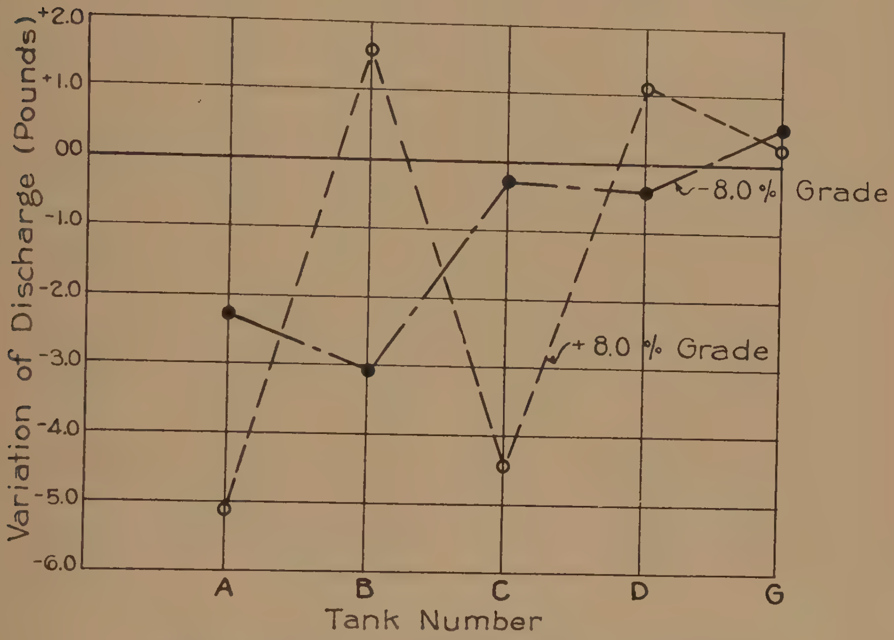

Figure 4. Effect of Grade on Quantity of Discharge

Note: Discharge at test setting with 85 lbs. pressure and zero grade as follows:

A—20 gallons
B—25 gallons
C—24 gallons
D—25 gallons
G—30 gallons

5. Effect of combination of grade and superelevation upon the quantity of water discharged.
6. Time required to fill tank at various pressures.
7. Time required to discharge various quantities of water.

For the test each tank was mounted on a segmental frame which permitted adjustment assimilating effects of grade either plus or minus, and effects of superelevation to right of left. Water was supplied at the desired pressure by two centrifugal pumps operated singly or in series. Test runs were made at pressures from 30 to 155 pounds as indicated by a gage at the measuring tank. The grades used were 0, 4 and 8 per cent plus and minus. The superelevations used were 0, 4, 8, and 12 per cent right and left.

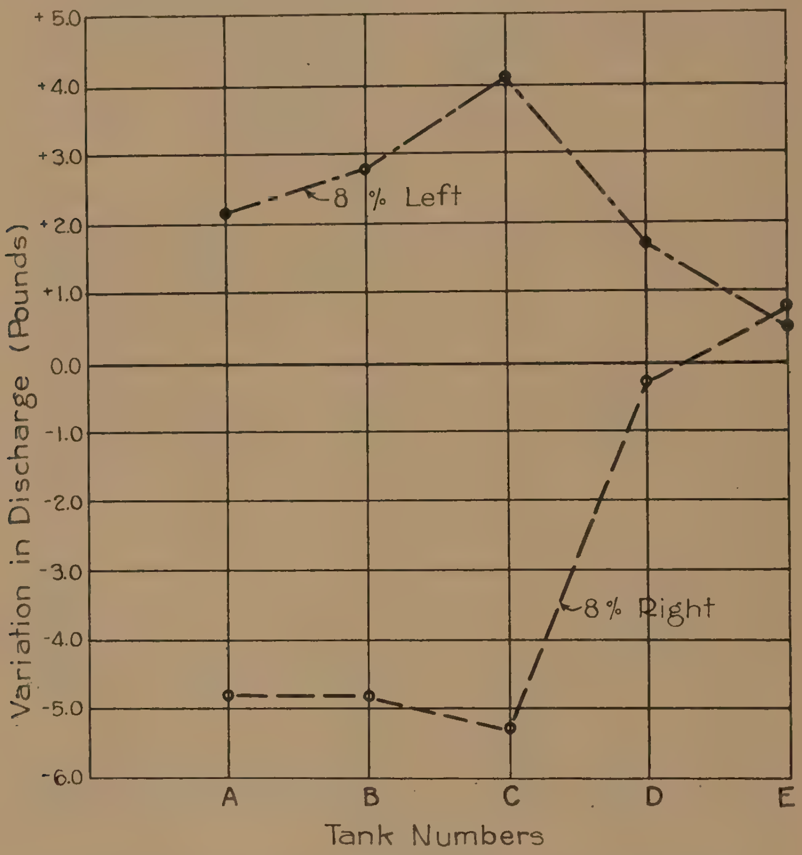

Figure 5. Effect of Superelevation on Quantity of Discharge

Note: Discharge at test setting with 85 lbs. pressure and 0.0 per cent superelevation as follows:

A—20 gallons
B—25 gallons
C—24 gallons
D—25 gallons
E—30 gallons

A plus grade was considered as raising the charging end of the paver. Right superelevation was considered as raising the right side of the paver, facing the direction of travel.

The indicator of every tank examined was found to be in error by various amounts. Variations of pressure were found to have a direct effect on the quantity of water discharged at different settings. Grades and superelevation separately and in combination affected the quantity of water discharged. With the piping arrangements and the pressures used the tanks filled and discharged within the customary mixing cycle. A summary of the effects of various factors studied is shown in Table I. A portion of these data are shown graphically in figures 4 to 7.

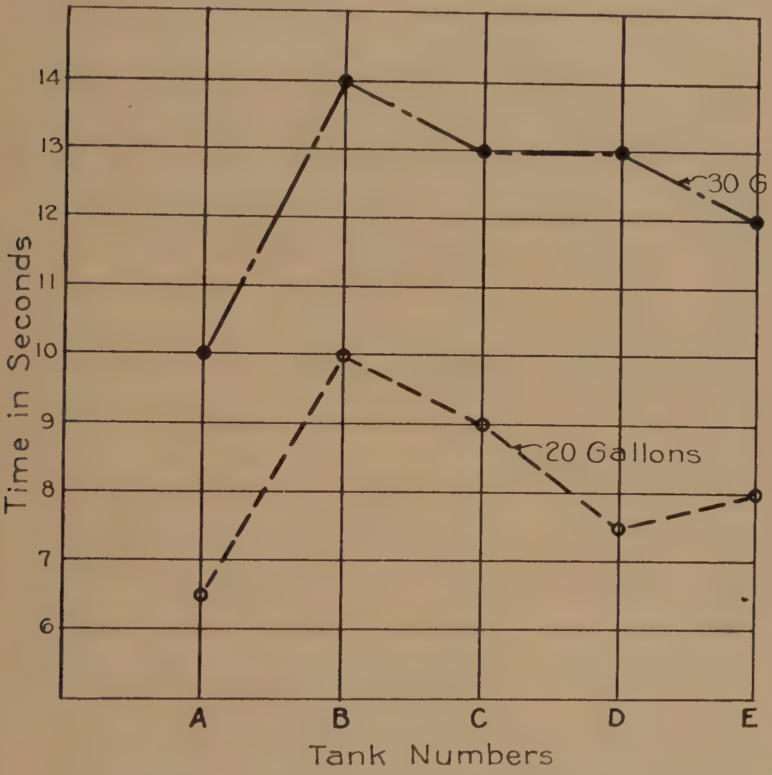

Figure 6. Time of Filling. Pressure 85 lbs. ±

It will be noted that the studies made by the University of Minnesota were conducted under controlled laboratory conditions. The equipment studied was furnished by manufacturers for the purpose of this study. It should also be noted that this study did not include that of the effect of air entrapped in the water supply.

The work of the Iowa Highway Commission was a study of the accuracy of water measurement on paving mixers in operation in the field. The measuring equipment had all been in use under field conditions for some time. A two-inch water meter of the oscillating disc type was connected in the water supply line near the measuring tank. This meter was protected by strainers. In the water line near the meter and between the meter and the pump were placed an air relief valve and a pressure gage. The air relief valve was designed much like the ordinary steam trap. This valve removed the air which was present in the form of large bubbles, but it probably did not remove all the small air bubbles.

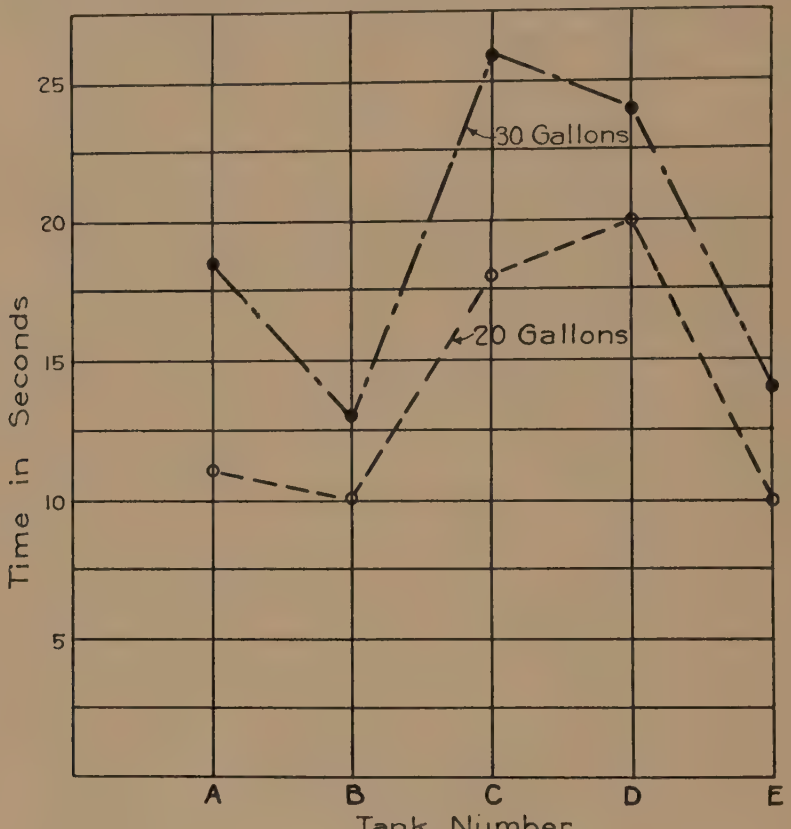

Figure 7. Time of Discharge

Note: Tank "D" discharge
20 gallon setting
169.5 lbs. in 20 sec.
171.5 lbs. in 25 sec.
173.7 lbs. in 30 sec.
30 gallon setting
252.0 lbs. in 24 sec.
253.7 lbs. in 30 sec.
254.9 lbs. in 60 sec.

Tank "G" discharge
20 gallon setting
Pressure 29½ lbs 55 sec.
Pressure 83½ lbs. 33 sec.
Pressure 152 lbs. 25 sec.
30 gallon setting
Pressure 29½ lbs. 81 sec.
Pressure 84 lbs. 50 sec.
Pressure 150 lbs. 38 sec.

TABLE I

SUMMARY, EFFECTS OF VARIOUS FACTORS ON ACCURACY OF WATER MEASUREMENT

MINNESOTA TESTS

Tank No.	Time to fill	Time to empty	Dial accuracy gal.	Discharge lbs. increase per lb. increase in pressure	Effect of + grade	Effect of — grade	Effect of R. super-elevation	Effect of L. super-elevation	Condition for minimum discharge	Condition for maximum discharge	Maximum difference
A	OK	OK	.9	.034	+	+	+	—	+8% grade 12% R.super. 30# pressure	—0% grade 12% L.super. 154# pressure	2.59
B	OK	OK	—	.011	+	—	—	+	—8% grade 12% R.super. 30# pressure	+8% grade 12% L.super. 155# pressure	2.17
C	OK s	slow	1.91	.023	+	—	—	+	+8% grade 12% R.super. 30# pressure	—8% grade 8% L.super. 155# pressure	2.07
D	OK	slow	0.3	.016	+	—	—	+	—8% grade 8% R.super. 30# pressure	+8% grade 8% L.super. 155# pressure	0.625
E			0.35								
F	OK	OK	0.52	.016	+	—	—	+	—8% grade 8% R.super. 15# pressure	+8% grade 8% L.super. 30# pressure	2.54
G			.77	.029	0	0	+	+	—	—	0.10

+ Discharge increased.
— Discharge decreased.
OK Time is short enough so that it does not slow up the mixer operation.

Observations were made on seven different mixers of three different makes in actual operation in the field. The period of observation for each mixer was one-half of a working day. The amount of water recorded by the meter was observed constantly. A record was made of the pipe line pressure at various times during the cycle of operation for each batch of concrete.

Starting with the measuring tank full, the discharge valve closed and the intake valve open, it was observed that when the position of the valves was reversed, the meter recorded a flow of water varying in quantity from 0.1 gallon to 3 gallons. This indicates that water was flowing from the supply line into the mixer drum during the time both valves were partly open. The total quantity that flowed through the valves during their operation varied for the make of mixer and for different mixers of the same make.

The detailed results of various tests are shown in Table II.

TABLE II

TESTS OF WATER MEASURING EQUIPMENT IN FIELD
IOWA TESTS

Mixer No.	No. of batches measured	Average leakage, gals.	Ave. max. pressure, lbs.	Variation metered batch from calibration setting			Total water per batch		Range of var., gals.
				Max.	Min.	Ave.	Ave. gals.	Max. var. from ave.	
1	66	0.305	101.0	3.63	0.23	1.543	35.305	2.245	4.00
2	29	0.466	40.0	4.3	0.2	1.641	37.766	3.034	5.50
3	91	0.617	75.8	2.8	0.0	0.866	34.516	2.884	5.60
4	152	1.354	106.6	7.28	0.1	3.691	36.230	4.430	8.70
5	93	Indeterminable	131.6	2.50	0.25	0.663	35.142	3.408	6.75
6	46	0.315	36.3	3.45	0.35	1.933	31.945	2.755	3.90
7	135	0.811	73.5	6.40	0.0	0.949	34.084	5.216	9.10

The results of both of these investigations emphasized the need for improvement and refinement of the design and construction of equipment used for measurement of water for concrete mixers.

With the indicator arrangements furnished, it is necessary that each tank be calibrated on the job before the original calibration may be accepted as correct. This criticism applies to all tanks tested during this study. It would seem that this error could be easily corrected by the manufacturer and is worthy of consideration.

One prominent defect of the measuring devices observed was in the three-way valve, unless this valve was automatically controlled. The mixer operator could inject into the mixer drum any additional amount of water, over and above that set on the indicator, by holding the valve lever in an intermediate position between the " fill " and "discharge" positions. This was possible because both intake and discharge valves were open at the same time and an unmeasured quantity of water was passed from the supply line directly to the mixer drum. With rapid operation of the valves this unmeasured quantity is small, but the quantity increased with slowing of the valve operation and increase in pipe line pressure. This same criticism applies to connected gate valves not so interlocked as to prevent one valve opening before the other is completely closed.

The quantity of water delivered to the drum was also affected by superelevation and grade. This effect was much greater in tanks mounted with the axis horizontal than for tanks mounted with the axis verticle. This is because it is practically impossible to design an adjustable draw-off or displacement device which will provide for the delivery of a constant quantity of water from a tank whose axis is horizontal for all normal positions of the tank and all settings of the control device.

The effect of variations in pipe line pressure may be eliminated by interposing a supplementary tank open to the atmosphere from which the water will be fed to the measuring tank by gravity. The air entrapped in the measuring tank may be removed by substituting for the air relief valves a riser pipe with a height equal to that of the top of the supplementary tank.

Such an arrangement of supplementary tank open to the atmosphere from which the measuring tank was fed by gravity and with a riser pipe substituted for the air relief valve was used on about 60 paving mixers in Iowa during 1929. The flow of water from the pipe line into the supplementary tank was controlled by a float valve. The flow of water from this supplementary tank to the measuring tank and from the measuring tank into the mixer drum was controlled by the ordinary three-way or connected gate valves. This arrangement was very satisfactory in every respect. Figure 8 shows a diagrammatic sketch of such an arrangement.

This arrangement not only allows air which is entrapped in the water supply to escape as it is discharged into the upper tank, but it prevents the possibility of air being held in the measuring tank due to faulty operation of air relief valves. This arrangement also relieves the control valves from excessive pressure which is a frequent source of trouble.

Since it would be very difficult to arrange any method for measuring the amount of water actually delivered to the mixer drum while the paver is in operation without hindering the work, such data are not available. However, it was the opinion of all engineers, inspectors and contractors interviewed that this equipment gave much more uniform measurement of water than did the ordinary single closed tank.

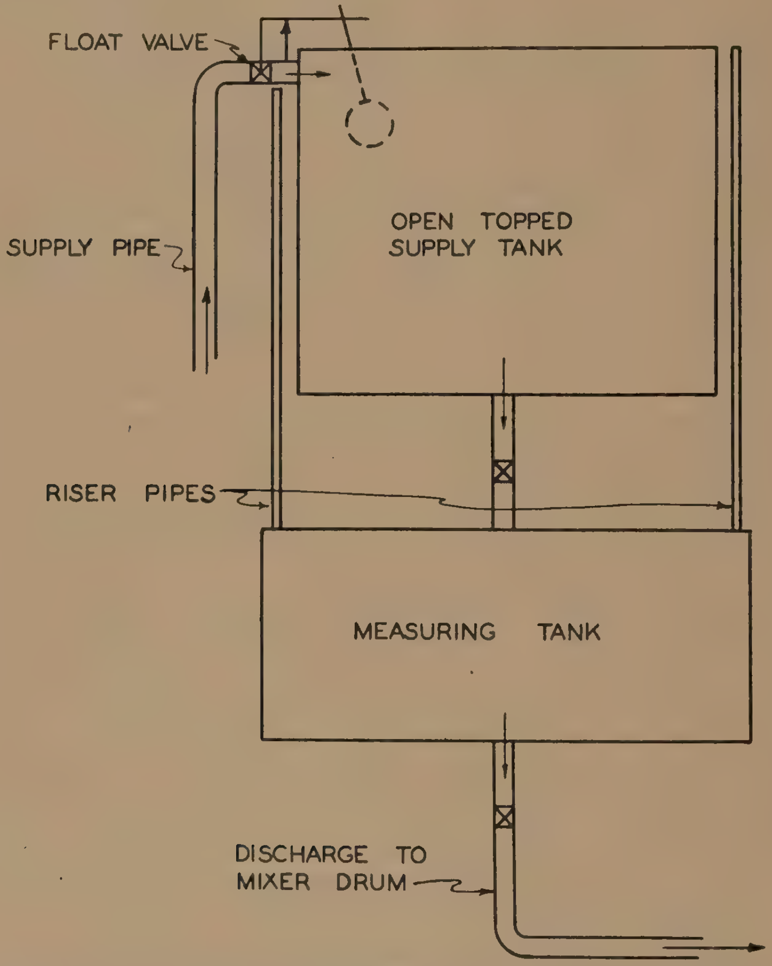
FLOAT VALVE
SUPPLY PIPE
OPEN TOPPED SUPPLY TANK
RISER PIPES
MEASURING TANK
DISCHARGE TO MIXER DRUM

Figure 8. Diagrammatic Sketch Showing Volumetric Measurement of Water under Atmospheric Pressure

The only difficulty encountered was that the float valve controlling the flow of water into the supply tank sometimes failed to close. This caused water to overflow onto the ground. However, since this was a source of irritation to the contractor rather than a source of defects in the work, the results were not serious and the trouble was corrected promptly.

The leakage of three-way valves may be reduced by equipping the plunger with follower skirts which will not clear the intake port before the discharge port is covered and vice versa. Gate valves may be interlocked so that they cannot both be open at the same time. With these revisions, the designs of tanks studied would measure water with reasonable accuracy. Present design tends toward mechanical operation of the valves. This should promote uniformity of water measurement.

One other improvement in connection with the operation of valves that may be worthy of consideration would be to arrange the mechanism so that it would be difficult for the operator to introduce into the mixer drum a quantity of water in addition to the measured quantity or make it impossible to do this without the knowledge of the inspector.

In justice to the mixer manufacturers, it must be said that they have taken note of the results of these studies and have made many of the improvements suggested. Valves have been improved to prevent the passing of water directly from the pipe line into the mixer drum. Vertical tanks have been substituted for horizontal tanks. Indicating devices have been improved and several manufacturers are prepared to furnish mixers equipped with supplementary tanks so arranged that water is fed to the measuring tank by gravity and a riser pipe open to the air is substituted for air relief valves.

Improvements which will eliminate the principal sources of error in water measurement can be made on mixers now in use at a nominal expense. It is suggested that the highway engineer will find it profitable to devote some attention to securing such improvements. The enforcement of specifications requiring accuracy of water measurement will no doubt stimulate further improvements in the design of water measuring devices.

REPORT OF COMMITTEE ON MINERAL AGGREGATES

W. J. Emmons, *Chairman*

Director, Michigan State Highway Laboratory

The newly formed Committee on Mineral Aggregates, organized as a subsidiary of the Committee on Character and Use of Road Materials, held its first meeting last Tuesday. It understands its duties to include the definition of problems in the realm of aggregates, the formulation of research projects, the sponsorship of the prosecution of these projects of cooperating organizations, and finally,the correlation and reporting of results. Its aim is to supply those who write specifications, with data upon which to base their requirements.

Permissible limits of test values of aggregates for a given use differ widely throughout the country. This is inevitable, and is many times a matter of economic necessity, since it is obvious that materials possessing similar characteristics are not universally available. Highways must be built, and aggregates, local or imported, must be used.

Even though there is logical reason for variation, the certainty exists that there is a serious lack of information regarding the true significance of many of the tests commonly specified. As a matter of fact, it is felt that many tests do not directly measure the value of the material for the use to which it is to be put. In addition, there is more than a suspicion that some important properties of aggregates are not measured, directly or indirectly, by present tests. It must, therefore, be admitted that present limitations of knowledge prevent the formulation of thoroughly intelligent aggregate specifications, which, in their ideal form, should combine the maximum of tolerance in the matter of test requirements, with the necessary degree of safety.

Although aggregates have been the subject of much study, very little has been accomplished in determining the effects of native characteristics upon their usefulness. A list of these characteristics includes hardness, toughness, strength, absorption, surface characteristics, density, chemical and mineralogical composition, and several others. They are inherent in the material and cannot be changed, but we do not know how to evaluate them accurately in terms of highway serviceability.

Anything like a complete investigation of these several properties, and their respective influences on the behavior of aggregates in all classes of highway work, would be a colossal task. The multiplicity of variables, which cannot be eliminated in the study of any one characteristic, indicates the necessity of extended and costly research, and probably the ultimate analysis of results by statistical methods. For instance, in studying the effect of toughness, it would be impossible to assemble a variety of materials, alike in a dozen other properties, and differing alone in toughness. Despite the extreme difficulty and magnitude of the problem, the significance of these inherent characteristics is fundamental and must sooner or later be the subject of intensive study.

Certain other characteristics are to a greater or lesser degree controllable, and form the bases of much uncertainty to the writer of specifications. Among these are gradation, shape of particle, and mineral or chemical coatings or incrustations. These require immediate attention, and present somewhat less difficulty to the investigator.

Therefore, while alive to the extreme desirability of instituting research with respect to the native qualities of aggregates, the committee, in the earlier stages of its activities, has decided to concentrate its attention upon several factors of the more controllable type. Plans have been made to outline investigations which will, it is hoped, clarify points over which there is much discussion. The general subjects of some of these are (1) the relation between the durability of coarse aggregates and that of mixtures, portland cement or bituminous, in which they are used; (2) the practical significance of coatings existing on aggregate; (3) the effect of characteristics of fine aggregate, other than gradation, on bituminous or cement mortars.

The inauguration of these few projects, important in themselves, will serve to launch the work of the committee. Laboratories, properly equipped, must be found to carry on the work. Willing cooperation in this respect is anticipated. As time goes on, and additional studies can be outlined, the scope and magnitude of the committee's work in this important field will be enlarged.

REPORT OF COMMITTEE ON HIGHWAY FINANCE

HARRY J. KIRK, *Chairman*

Consulting Engineer

In the early nineties William McKinley, then Governor of Ohio, appointed a commission to investigate the feasibility of improving a state system of " wagon roads." The commission, after careful study, reported against the proposition. It must be remembered that those were the days of the " trolley car " boom. Insignificant indeed was the town which could not boast of several such lines projected into it. And the commission reflected this state of affairs as the principal reason for its adverse report. In substance it reasoned that inasmuch as there would soon be a net work of trolley lines to furnish ample transportation, there was no need for the improvement of " wagon-roads."

That was less than forty years ago. The " trolley-car " and the wagon are almost curiosities on our highways today. The invention of a new machine, the motor vehicle, ushered in a new era in highway transportation, requiring new kinds of roads and these roads in such quantity as to make highway financing a greater problem than ever before in history.

The rapid development of this new vehicle has called for profound changes in road design and road building methods. Its further inevitable development will require further revision in road design and road building methods. It is pointed out here with emphasis that the rapid development of the motor vehicle has aspects which have a vital bearing on highway finance policies.

In keeping with the general plan of work for the year the Committee on Highway Finance has limited itself chiefly to the preparation of a program of research.

SCOPE, LIMIT AND SYNOPSIS

The fundamental problem in highway finance is two-fold, pertaining first to the sources from which funds shall be collected and second, to how these funds shall be distributed.

We have chosen as our scheme of attack in outlining this program of research to discuss this question from five separate aspects, namely those of the road user, the abutting property owner, the community, the state and the nation. Some space is also given to the question of the use of credit in highway financing.

THE ASPECT OF THE ROAD USER

The road user is a comparatively new contributor toward road costs, but an increasingly important one. With the abolition of toll roads the road user ceased to pay anything directly toward the country's road bill. Today he is most important and is constantly increasing his contribution. The motor vehicle registration fee first put into effect a quarter of a century ago was for fifteen years the sole method of making a charge directly against the road user. Today it is superseded in importance by the gasoline tax.

The popular notion that road users should pay for roads and that the gas tax is an equitable road usage charge is the principal reason for its rapid development. The low cost of collection, the difficulty of evasion and the " painless method of extraction " have all aided in making it as popular as any tax can be.

The conflict in legislative halls has generally *not* been over the question as to whether there was or was not to be a gas tax, but rather as to how the proceeds were to be distributed.

The equitable distribution of user taxes is one of the most pressing problems of highway finance today.

Proposed generally as a means of building a state road system, the history of most gasoline tax measures will reveal that part of the proceeds have had to be allotted to local governmental units for road and street purposes in order to get enough votes to make the measure a law.

Benefits to the Road User. Fundamental questions are, how much *is* the road user benefited, and how much of the highway bill should *he* pay?

There has been a great change in popular opinion on this subject. Fifteen years ago there was still a large school of thought which maintained that the road user should pay little, if anything, that the great bulk of highway revenues should come from general taxation. This stand was maintained on the broad principle that good roads were of wide spread benefit.

The next step was the idea that general taxation should build the roads and that the road user, through registration fees, should maintain them.

Since the advent of the gasoline tax in 1919 the share contributed by the road user has grown by leaps and bounds. In 1921 "user taxes" constituted 12.3 per cent of the total spent for rural highway purposes while in 1928 this figure was 44.6 per cent. Expenditures on municipal streets are not included in the figures just given.

Mr. Chas. H. Upham in the proceedings of the American Road Builders Association for 1929 says:

> "The advent of the gasoline tax and registration taxes has solved the problems of state highway taxation to a large extent. Inasmuch as 75 per cent of the total expenditures on state systems is derived from gas tax and license fees at the present time, it would seem a possibility that these might become the absolute source of revenue in due time."

User Taxes and Municipal Streets. The proponents of state highway improvement look with alarm on the growing demand, particularly on the part of municipalities for a share in gasoline tax funds as a menace to the early completion of state highway systems.

The principal argument brought forward by city proponents is that inasmuch as a large proportion of motor fuel is consumed on city streets, so too a large proportion of the gas tax should be returned to municipalities for street repair and improvement.

Let us digress a moment and examine into the validity of the argument.

Up to this time we have not seen any figures published purporting to show the proportion of gasoline consumed on city streets. The chairman of this committee did make such a calculation a few years back. Based on transport survey data and certain well taken assumptions the result indicated for the State of Ohio covering the year 1925 that about 43 per cent of the gasoline was consumed on municipal streets. Of course this figure would vary in various states depending on area and distribution of population as between cities and rural districts.

User Taxes and Rural Highways. It would seem, however, in considering the subject of financing rural highways, whether state or local, that the relative amount of gasoline consumed on city streets and rural highways is not of primary significance. Rather the thing of fundamental importance as regards rural highways is the relative proportion of usage of these highways by city owned and farm owned vehicles. We present, herewith, data on this point gathered in transport surveys conducted jointly by certain states and the U. S. Bureau of Public Roads. The figures are for the State Highway System in Ohio which constitutes 13 per cent of the total rural mileage of that state, New Hampshire where the corresponding figure is 11.3 per cent and for Vermont where the figure is 30.6 per cent. The data given show the per cent of total vehicle miles on the State Highway Systems of these states as divided between city-owned and farm-owned vehicles.

Per Cent of Vehicle Miles

State	Autos		Trucks	
	City-Owned	Farm-Owned	City-Owned	Farm-Owned
Ohio	87.6	12.4	84.5	15.5
New Hampshire	93.9	6.1	88.6	11.4
Vermont	89.9	10.1	81.1	18.9

In commenting on these figures at the July, 1929, meeting of the American Society of Civil Engineers, Prof. H. R. Trumbower said:

> "These figures and the results of other traffic surveys show that the high type improvement and maintenance of state highway systems is, therefore, primarily the result of the demand for highway service by city motor vehicle owners. It should also be borne in mind that on these state highway systems of the country, which constitute approximately ten per cent of the total rural mileage, about one-half of the total rural highway expenditures are made."

The Road User and State Highways. Data on rural highway usage indicate that the quickest and largest return for the funds expended can be made to the road user by the improvement of the state trunk highways which, considering their mileage, carry an overwhelming proportion of the traffic.

For example the Ohio Transport Survey indicated that the state system, comprising but 13 per cent of the total rural highway mileage, was carrying 57.7 per cent of the traffic measured in vehicle miles. Contrasted with this are the local or township roads, 59.9 per cent of the total rural mileage carrying only 12.9 per cent of the total vehicle mileage. These figures showed that mile for mile state roads were used four times as much as those on the county systems and twenty times as much as on the township systems.

Amounts, Methods and Distribution of "User Taxes." A careful study of the methods of imposing gasoline taxes and registration fees and the provisions for the distribution of the revenues derived therefrom throughout the various states indicates no agreement as to the one or the other.

It should be comparatively simple, however, to point out the more efficient methods of collection.

In spite of the maze of differences apparent in state laws there certainly must be some underlying principles which should govern the amount of these " user taxes " and the way the proceeds should be distributed among the different classes of roads.

Needed Research

1. What principles should govern the distribution of revenues from the gas tax and registration fees?
    - (a) What proportion if any should municipalities receive and what limitations should be placed on municipal use, such as, restricting expenditures to maintenance, or requiring a certain minimum proportion to be expended on through arteries?
    - (b) How much should be allotted to county roads? To purely local roads? And should the state exercise supervision over the expenditures of funds so assigned?
    - (c) To what extent should state highway programs be financed by " User Taxes "?
    - (d) What is the relation, if any, between the percentage of city-owned cars and farm-owned vehicles and the corresponding proportion of highway usage by these classes?
2. What principles should govern the amount of funds raised by gasoline taxes and registration fees?
    - (a) Should the collection of all motor vehicle taxes be reserved to the states exclusively?
3. To what extent is it advisable to enlarge state control over rural highways? (This is also noted under " State-wide Aspect ".)
4. Are there circumstances under which the repeal of property taxes on motor vehicles is justified?
5. What are the best methods of administering the collection of registration fees? Of gasoline taxes?

THE ASPECT OF THE BENEFITED PROPERTY OWNER

As a general rule abutting and adjacent property is always benefited by street or highway improvement, be it a purely local road or a main artery. The benefit, however, is not necessarily in proportion to the cost of the improvement, particularly in the case of trunk routes which have to be built heavy and wide. Studies reported by the U. S. Bureau of Public Roads covering farm property located on mud roads in strictly rural areas indicate that such property is considerably increased in value by the provision of a road improved to the minimum of furnishing an all-weather surface. However, the increment in value is very little for higher type improvements under the same conditions.

There was a time when the adjacent property owners paid almost the full bill for road and street improvement; but, just as with the growth of motor vehicle traffic state and federal participation has been made necessary, so there is a growing demand that community participation be correspondingly increased on local road and street work to still further take over the burdens carried by abutting property.

In cities and built-up communities the value of property will continue to far overshadow the cost of street improvement as compared with farm values, and the expense of rural highways. Hence, regardless of " benefits " the " ability to pay " will mean that, property assessments will still continue to be a principal factor in financing municipal street improvements.

In rural areas the problem is a difficult one. The slump in farm values, beginning about 1920, and the apparent low level of profits from agricultural pursuits has made a great change in the attitude of rural residents toward property assessments. Here again " ability to pay " must be given consideration along with the estimated " benefits."

NEEDED RESEARCH

1. Studies in various localities to determine the effect of road improvement on property values.
2. Studies to determine the type of road adequate for local needs and the point at which further improvement ceases to proportionately benefit land values.
3. Studies to determine the effect on land values of distance from markets.
4. An analysis of the above studies to evaluate the amount of assessment that may be justly made against (1) farm property in rural areas, and against (2) property located in or adjacent to centers of population when (a) on state trunk highways, (b) on county or community roads and (c) on local roads.

THE COMMUNITY ASPECT

Trunk Routes. Benefits to a community from road improvement will vary widely. In thickly populated areas community benefit may so far overshadow the combined state and national benefits, even on trunk routes, as to constitute the major part.

For example, a trunk line route through a rural area may carry twenty-five per cent of foreign or out of state traffic. As one approaches centers of population the total traffic rapidly increases due to the added increment of community transport. Hence, it is that the percentage or through traffic diminishes in urban areas so that in the larger municipalities it is but a fraction of one per cent.

If taxes are to be paid according to benefits, and the benefits in such cases parallel usage, the community contribution toward trunk line highway costs will increase as the population density increases. On this theory municipalities, particularly the larger cities, should continue to pay practically the whole bill for city streets, even where these carry the through traffic. A contribution by the state might be justified on trunk routes in smaller communities.

Local Roads and Streets. Under local roads and streets we might classify all those routes of travel not a part of the state or federal systems. These local highways can logically be placed in two divisions, namely, those which chiefly serve the residents living thereon and secondly, those which play a larger part in carrying community traffic, such as a county highway system or the principal streets of a municipality. In all such cases the benefit to the community and to adjacent property contingent on highway improvement is proportionately large and the state-wide or national benefits are negligible.

It is a certainty, however, that in areas of low wealth, unless financial aid is received from some outside source, local roads cannot be improved. That is a condition of fact and no amount of theory about benefits is going to change it. " Ability to pay " will have to be considered here. And the outside help will have to come from a source more " able to pay " than the poor areas.

The Local Road Problem. The magnitude of this problem as it affects rural highways may be realized from the following figures:

Local governmental units have responsibility for 90 per cent of our rural highways, or 2,700,000 miles. Of this vast mileage only 15 per cent is surfaced, and the present annual rate of improvement is about one and one-half per cent.

Of course many miles of such roads will likely never be surfaced in any period we can estimate; but, nevertheless, it is a very serious problem where vast areas are deprived at least for many months in the year of what in this swift moving age and in these United States has already become the only efficient means of highway transportation, the motor vehicle. It is an outstanding bit of evidence to note that in the decade preceding 1920 the rural population in eighteen states actually decreased. For the country as a whole during this same period urban population increased 20 per cent and rural population increased only 3 per cent. At this rate the coming census may show an actual decrease in rural population for the present decade. This would indicate that traffic on local rural highways will increase very slowly, if at all. It might suggest the advisability of the abandonment of many such back roads.

Needed Research

1. Studies of economic changes in typical communities to show the benefit to the community of
    (a) Trunk line highway improvements.
    (b) County highway facilities and arterial streets.
    (c) Purely local roads or streets.
2. A study of the forms of road administration in local governmental units, together with the methods of raising and spending road monies, and an analysis to point out the most efficient forms and methods in local road administration.
3. An analysis of the problem of financing improvements on local roads in areas of low wealth with suggestions for its solution.

THE STATE-WIDE ASPECT

The principle is well established that there should be a state system of highways financed mostly, if not wholly, out of state funds. There was a time when tax levies on real property were used to a large extent to provide funds for state highway construction, but such levies have largely given place to motor vehicles or " user taxes." It would seem logical, however, that as federal appropriations for highway purposes are made out of the general funds used to maintain the United States Government, by the same token and to a larger degree state funds might be appropriated for state roads from the same sources used to support the other branches of state government.

Extension of State Control. There is a school of thought gaining rapidly in influence which advocates the extension of state control over rural highways. In some states this takes the form of a state-aid system in addition to the state trunk highways.

A share of state " user taxes " is often allotted to local governmental units with no provision for state supervision of its expenditure. Some very solid arguments are advanced for the exercise of some sort of state control, particularly where efficient supervision and management cannot be provided locally.

Needed Research

1. On the theory of benefits what proportion of state highway revenues should come from state general funds?
   (a) A study of actual cases where state general revenue funds have been used for highway improvement both on state roads and local roads, bringing out the circumstances under which the method was adopted, the amount of the appropriations, and if finally abandoned, the causes therefor.
   (b) The arguments pro and con on " user taxes " as a fair measure of state-wide benefits, with data from those states confining the state's contribution exclusively to " user taxes."
2. A study of what constitutes a state road in various parts of the country, and an outline of those principles or conditions which should qualify a road to be a part of a state system. Such a study should include the question of state-aid on local roads and the advisability of extension of state control.

The facts brought out in recent transport surveys will shed much light on these questions.

NATIONAL ASPECT

In these days of country wide travel there is a benefit from highway improvement more universal than ever before in the history of highway transportation. From this standpoint the use of federal funds in road work is amply justified.

Needed Research

1. A study bringing out to what extent the federal government should participate in the total cost of highways, and in the cost of individual highways.

It is suggested that an analysis of data available from traffic studies, such as length of trip, kind of business, status of ownership, etc., will be of material help in such a study.

An inquiry of various state highway departments and other sources throughout the country would bring out facts and experience very pertinent to the above questions bearing on federal participation in highway work.

THE USE OF CREDIT

We now come to a matter of policy in highway finance that is indeed a live question, the use of bonds for highway purposes. Platt Young in Georgia Highways magazine puts it this way:

> "To bond or not to bond? The query flies
> From lip to lip, and brings its apt replies:
> Some reckon wisely, others frankly own
> The problem has a weight that vies with stone,
> If roads are commerce lanes for faring man,
> Who rides must pay, still proves the commerce plan;
> But, if the future sons shall pay the toll
> 'Twill make them glimpse a long-encumbered roll.
>
> "To bond or not to bond? The question here
> Bobs up and down throughout each passing year,
> And while the tide of time still onward flows
> Its shores are laden with men's ayes and noes."

Several important studies have been made in the last year or two analyzing the results of the use of bonds in state highway work.[1, 2] They indicate, that, where large mileages of state trunk highways potentially of great general use are now unimproved, the use of bonds, thus quickly obtaining enlarged highway facilities, will return handsome dividends in reduced transportation costs. Under such conditions the use of credit is amply justified from an economic standpoint.

The studies of state systems here referred to justify the bond issue method principally on the estimated saving in transportation costs. And this argument is irrefutable, particularly where mud roads on trunk lines are replaced by well located smooth highways adequate at all times of the year.

Term of Bonds. There is lack of agreement on the length of term for highway bonds. Twenty and thirty year terms are frequently

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[1] A. J. Brosseau, "Highway Finance in U. S." International Chamber of Commerce, Amsterdam, July 1929.

[2] Chas. H. Upham, "Highway Finance." Proceedings of American Road Builders Assoc. for 1929.

justified on the ground that a large per cent of the original investment represents permanent value. The validity of this argument for long term road bonds is open to serious question.

The rapid development of the motor vehicles has called for a corresponding rapid development in road design. Many roads constructed to the best standards of ten years ago or more are today inadequate. This is due principally to the great increase in the speed of motor vehicles over that of a decade or more ago. Ten year old pavements are too narrow, road crowns are too high, curves are too sharp, bridges are not wide enough, and surfaces are entirely too rough for today's high speed vehicles. There are many miles of pavements and numerous bridges still capable of long service as far as the materials in them are concerned, located in states which started their road programs early, which today must be completely replaced and yet have little salvage value. Many examples of this kind, in age much less than the term of highway bonds used in many states, may be cited. In view of these lessons from the past it is most opportune to ask, " What of the future ? "

Will the promotion and development of the motor vehicle as to numbers and speed, perhaps width and weight, continue to grow in the next ten years as it has in the past ? If so, we are going to find before many years roll 'round that the highways of today have become as hopelessly out of date as those built during war times appear to us now. Their salvage value will be but a fraction of what we estimated and the bill for their cost will in a large part be yet unpaid.

Needed Research

1. Studies of economic changes in states that have used the bond issue plan and others that have used the so-called pay-as-you-go plan.

   Similar studies might be made in communities financing local road programs.
2. Studies to determine the proper term of bonds in years for use on the several classes of routes and various types of construction.

CONCLUSION

In closing emphasis is again placed on the value of the facts brought out by recent comprehensive traffic surveys, especially those carried out cooperatively by the U. S. Bureau of Public Roads, the various state highway departments and other units.

Right conclusions on the problems of highway finance cannot be reached by any one sided view of the subject. Due to the preëminence of state road programs there has been a tendency on the one hand to neglect the local highway problem and on the other hand to ignore the traffic congestion prevalent in urban areas. No one will dispute the unwisdom of one sided development of road facilities.

It is well also to keep in mind that our highway budget is in competition with other much needed public expenditures. Many public improvements have been made, and highway improvements are no exception to the rule, not because they were the most needed, but because the people behind them were better politicians or more skillful at moulding public opinion.

It would be a wise step forward if our governmental units, federal, state and local would plan their capital expenditures, including highway expenditures, over a long period of years. Thus it would be found that the funds available would be spent more efficiently and to the very best interest of all the public.

REPORT OF COMMITTEE ON HIGHWAY TRANSPORTATION COSTS

T. R. Agg, *Chairman*

Professor of Highway Engineering, Iowa State College

This report consists of four sections. The first presents a method for determining the true annual cost of any specific highway as of the date when the data are assembled, including an annual charge predicated on future periodic reconstruction. The second section presents a method for estimating the annual cost of a highway as of date when data are not available for computing the true annual cost. The third section presents a method for determining the cost of vehicle operation that is applicable to automobiles and commercial vehicles. The fourth outlines a method of calculating the cost of highway transportation per vehicle mile of road use, by applying the data of sections I or II and III.

SECTION I

Comprehensive Method of Computing the Annual Cost of Highways

The annual cost of a road (not road value) may be expressed as the total average yearly expenditure that will construct, replace and maintain in perpetuity in standard serviceable condition any existing road under existing traffic and climatic conditions. This amount may be calculated either by method (A) or method (B).

Method (*A*): By determining the amount of money which if set aside today will return in perpetuity as interest, sums sufficient to pay annual interest charges on construction cost, to provide a sufficient annual maintenance charge, and to accumulate periodically necessary replacement costs; and by multiplying that amount by the rate of interest prevailing in current state financing.

Method (*B*): By determining directly the aggregate of the annual cost of each of the items entering into the cost of constructing a road and replacing and maintaining it indefinitely, or for a very long period.

Method (A) may be put in terms of a formula as follows:

$$C = \text{Average annual road cost} = \left(\begin{matrix}\text{cost to}\\ \text{construct}\end{matrix} + \frac{\text{yearly maintenance}}{\text{rate of interest}} + \frac{\begin{matrix}\text{periodic maintenance}\\ \text{(reconstruction)}\end{matrix}}{(1 + \text{rate of interest})^n - 1} + \cdots \right) \text{times rate of interest}$$

$$\text{that is, } C = r\left[A + \frac{B}{r} + \frac{E}{(1+r)^n - 1} + \frac{E'}{(1+r)^{n'} - 1} + \text{etc.} \right] (1)$$

Where

C = average annual road cost.

A = cost to construct.

B = yearly maintenance cost (every year).

E (or E') = expenditure for periodic maintenance every n (or n') years. (Replacement is an "E" value.)

r = the rate of interest prevailing in current state financing.

While the basis of the computations is relatively very simple, the actual computations may be very much involved because of the great number of items to be taken into account. Unless the records relating to any particular road are complete, it will not be possible to obtain the data for exact computations, but many of the state and municipal highway departments do have good records of the more recent projects.

The desirability of special consideration and separate statement of the expenditures for grading, drainage structures, and right of way is emphasized.

On account of the fluctuations of the value of money, the cost can only be compared as of a specific date by applying the data from the Engineering News-Record price index numbers, or other reliable source.

The items of information needed for the determination of highway cost are shown in Figure 1. The actual cost of each item is to be determined and then calculated as of date. Annual road cost is then computed by applying the sums thus obtained for each group of items in formula 1.

SECTION II

Method of Estimating Approximate Cost of Highway Transportation

The approximate cost of highway transportation may be estimated when data are not available for computation in accordance with the

- HIGHWAY TRANSPORTATION COSTS ON *X* HIGHWAY WHEN ANNUAL TRAFFIC IN BOTH DIRECTIONS CONSISTS OF . . . . AUTOMOBILES, COM. VEHICLES UP TO 2½ TONS CAPACITY . . . . COM. VEHICLES OVER 2½ TONS CAPACITY . . . .
  - VEHICLE
    - FIXED
      - Interest
      - Depreciation
      - Insurance
      - License and Property Tax
      - Garage
      - Supervision
    - VARIABLE
      - Drivers
      - Tires
      - Gasoline & Oil
      - Maintenance
  - HIGHWAY
    - "A" ITEMS — FIRST COST (capital costs)
      - Land
        - Location
        - Date of Acquisition
        - Cost
        - Length & Width
        - Ruling Grade
        - Soil
        - Climatic Conditions
      - Subgrade
        - Date of Construction
        - Total Expenditure
      - Base
        - Number of Courses
        - Type of Material
        - Date of Construction
        - Cross Section
        - Total Expenditure
      - Surface
        - Number of Courses
        - Type of Material
        - Date of Construction
        - Cross Section
        - Total Expenditure
      - Structures
        - Drainage
          - Number and Type
          - Date of Construction
          - Total Expenditure
        - Bridges
          - Length and Type
          - Date of Construction
          - Total Expenditure
        - Grade Separations
          - Length and Type
          - Date of Construction
          - Total Expenditure
      - Engineering and Administration
    - "B" ITEMS — ANNUAL MAINTENANCE COST BASED ON EXISTING TRAFFIC
      - Non-surface
        - Subgrade
        - Base
        - Shoulders
        - Roadside (trees, etc.)
        - Snow Removal
        - Bridges
        - Drainage Structures
        - Fences and Signs
        - Grade Separation
      - Surface
        - Painting
        - Filling Ruts
        - Patching
        - Blading or Dragging
      - Engineering and Administration
    - "E" ITEMS — PERIODIC MAINTENANCE INCLUDING REPLACEMENT COST BASED ON EXISTING TRAFFIC
      - Non-surface
        - Fences and Signs
          - Amount Expended
          - Interval in years between expenditures
        - Bridges
          - Amount Expended
          - Interval in years between expenditures
        - Drainage Structures
          - Amount Expended
          - Interval in years between expenditures
        - Grade Separations
          - Amount Expended
          - Interval in years between expenditures
      - Surface
      - Engineering and Administration

X—insert here the complete description of the highway to show length, type and termini.

Figure 1

exact method outlined in Section I. An estimate should be made for each type of construction of which a sufficient mileage exists to permit reasonably accurate computations of average costs per mile, or per square yard. The several items of cost to include are shown in Figure 2. Then equation 1 applies as before. Each of the costs outlined in Figure 2 should be determined from actual records of expenditures on the system of roads in question, or if records are not available, from the cost of like items on other roads. The calculations will be of little value unless data are available for use in determining the principal items, but some minor items may be estimated without influencing the results appreciably. The costs will in general be determined as follows:

Item 1

Right-of-Way. Assume the easement for the right-of-way to have a value equal to the value of the land for agricultural or other purposes at the present date.

Item 2

Drainage Structures. Use the actual cost of the drainage structures as of record except for major stream crossings that serve traffic from additional miles of road. The fair proportion of the cost of these major stream crossings to be charged against the system under analysis. The cost of major stream crossings is to be reduced to a cost as of a specific date by applying the Engineering News-Records price index, or other reliable data.

Item 3

Earthwork and Prior Surfaces. Charge the actual cost of grading and prior surfaces. By prior surfaces is meant any wearing surface that has become an integral part of the existing wearing surface, as when a surface is changed to a higher type.

Item 4

Road Surface. Determine from the construction records the actual total cost of the road surface and reduce to a cost as of a specific date by applying the Engineering News-Record price index, or other reliable data, to the sections constructed during each year.

- APPROXIMATE HIGHWAY TRANSPORTATION COST ON ANY ONE TYPE OF ROAD SURFACE BASED ON AVERAGE EXPENDITURES FOR THAT TYPE IN A STATED AREA (CITY, COUNTY OR STATE) FOR AN AVERAGE TRAFFIC OF.... AUTOMOBILES AND .... COMMERCIAL VEHICLES PER ANNUM
  - VEHICLE
    - FIXED
      - Interest
      - Depreciation
      - Insurance
      - License and Property Tax
      - Garage
      - Supervision
    - VARIABLE
      - Drivers
      - Tires
      - Gasoline and Oil
      - Maintenance
  - ROAD COSTS
    - "A" ITEMS FIRST COST (Capital costs)
      - Right-of-way
      - Drainage Structure
      - Earthwork and Prior Surface
      - Road Surface
      - Signs and Other Appurtenances
      - Administration and Engineering
    - "B" ITEMS ANNUAL MAINTENANCECOSTS BASED ON EXISTING TRAFFIC
      - Non-surface, i. e., Shoulders, Signs, Drainage Structures, etc.
      - Road Surface
      - Administration and Engineering
    - "E" ITEMS PERIODIC MAINTENANCE BASED ON EXISTING TRAFFIC
      - Date of Expenditure
      - Total Expenditure

Figure 2

Item 5

Signs and Other Appurtenances. Compute the total cost of signs, guard fence and similar appurtenances. Crossing eliminations existing on the system to be handled in the same manner as major stream crossings.

Item 6

Engineering and Administration. The cost of engineering and administration is to be determined by applying to the total cost of all of the foregoing items the percentage which represents the actual cost of this item in the jurisdiction. (5 per cent may be used as a close approximation.)

Item 7

Summary. The grand total of items 1 to 6, inclusive, constitutes the quantity A in formula 1.

Item 8

Maintenance Cost. The items of maintenance cost shall be determined from records of maintenance cost on the roads under consideration, supplemented by records of cost on like roads under equivalent traffic conditions in near-by areas where climatic conditions are similar. Where the types of surfaces require routine maintenance supplemented by periodic special maintenance such as resurfacing, reoiling and the like, the annual maintenance cost shall be determined as prescribed in Section I. The maintenance costs shall include the appropriate rental charge for equipment.

Item 9

Engineering and Administration on Maintenance. This shall be determined by calculating the ratio of such overhead costs to the total expenditures for maintenance in the jurisdiction, and applying that percentage to total of Item 8.

SECTION III

Annual Vehicle Costs

The following items comprise the cost of operation of a motor vehicle, which for convenience, are to be reported on the annual basis. There is appended a table showing a method of reporting these costs in such a manner as to permit the calculation of cost per mile.

A. FIXED COSTS

Item 1

Annual Interest Charge. This is obtained by computing the interest on the investment less accrued depreciation at an appropriate rate. (6 per cent suggested by the committee.)

Item 2

Annual Depreciation Charge. It is recommended that depreciation be computed as a uniform annual charge during the life of the vehicle. For automobiles the present information indicates that the average life will be about seven years and the depreciation per annum will be about 15 per cent of the initial investment.

Item 3

Annual Insurance Cost. Actual amount paid but when no insurance is carried, use the standard rate for " fire, theft, public liability and property damage."

Item 4

Annual license fee plus property taxes, if any are paid, but not including gas tax.

Item 5

Annual Cost for Garage. The actual rent paid or the rental value of the garage space in which vehicle is housed.

Item 6

Annual Supervision Costs. Applicable to fleets of vehicles that require supervision and management. The cost to be determined from accounts and prorated to each vehicle.

B. VARIABLE COSTS

Item 7

Annual Cost of Drivers. Actual amount paid for drivers.

Item 8

Annual cost of tire equipment, including repairs and replacements.

Item 9

Annual cost of gasoline and lubrication.

Item 10

Annual cost of maintenance, including all expenditures for the mechanical upkeep of the vehicle, cleaning and painting.

SECTION IV

Transportation Costs

The annual cost of any system of roads as determined by Method I, or the approximate annual cost as determined by Method II, is reduced to annual cost per mile.

The average cost of automobile operation per mile is determined as prescribed in Section III.

The cost of vehicle operation chargeable per mile of road is calculated by applying the average cost per vehicle per mile to the average yearly traffic on the road and by subtracting from this the sum per mile of travel that amount of traffic has paid into road funds through the gas tax and license fee.

Summary. On the basis of the foregoing, the following formula is proposed:

$$\left.\begin{array}{l}\text{Highway}\\ \text{Transportation}\\ \text{cost per vehicle}\\ \text{mile}\end{array}\right\} = \frac{\left\{\begin{array}{l}\text{Annual cost}\\ \text{of roads}\\ \text{per mile}\end{array}\right\} + \left\{\begin{array}{l}\text{Annual operating cost per mile}\\ \text{of annual traffic, less contri-}\\ \text{bution to road funds}\end{array}\right\}}{\text{Annual traffic per mile}}$$

This report undertakes to lay the general groundwork only for consideration of the subject. It indicates the several kinds of data that must be assembled in order to develop satisfactory quantities for use in the formulas proposed. Appended is a form proposed for use in securing certain data regarding the cost of vehicle operation together with an analysis of the items which enter into a complete statement of highway transportation costs covering both vehicle and highway charges.

FORM FOR REPORTING THE ANNUAL COST OF A MOTOR VEHICLE

Make .... Type Body .... Model .... Year .... When purchased .... 19 .... Capacity .... passengers or tons. Average load .... passengers or tons. Total mileage to date below .... Mileage for year .... % of mileage on Paved Road .... % of mileage on Unpaved Road ....

Annual Cost for Year Ending .... 19 ....

Fixed Costs

No.	*Item*	*Amount*	*Remarks*
1.	Annual interest charge on depreciated value..	$..........	
2.	Annual depreciation charge.................	$..........	
3.	Annual insurance cost.....................	$..........	
4.	Annual cost of license and taxes............	$..........	
5.	Annual rental cost of garage................	$..........	
6.	Annual pro rata supervision cost............	$..........	
Nos. 1-6.	Total annual fixed cost.............		$..........

Variable Costs

7.	Annual cost of drivers.....................	$..........	
8.	Annual cost of tires.......................	$..........	
9.	Annual cost of gasoline....................	$..........	
10.	Annual cost of lubrication.................	$..........	
11.	Annual cost of maintenance................	$..........	
7-11.	Total annual variable cost.................		$..........
1-11.	Total annual costs.........................		$..........
	Cost per vehicle-mile.....................		..........¢

Note.—All amounts entered should be determined by actual accounting, where possible. When estimated, note "est" after figures and explain basis upon which estimate is made.

Confidential

Owner ....

Address ....

Reported by ....

PROGRESS REPORT

SPECIAL INVESTIGATION ON CURING OF CONCRETE PAVEMENT SLABS

F. C. Lang, *Chairman of Committee*
Fred Burggraf, *Special Investigator*

The reports of 36 investigations (28 field and 8 laboratory) on curing of concrete pavements carried on in the United States during the past eight years have been studied. Field and laboratory research projects needed to provide additional information on the problem have been outlined. Some investigations, in accordance with this outline, are under way and several more are assured for the coming year. An intensive survey of the surface condition of about 3000 miles of concrete pavements has been about 75 per cent completed.

This report gives some tentative conclusions which have been drawn by the committee from a preliminary study of the data now available.

Definition and Function of Curing

Curing and results of lack of curing are summarized in a concise manner in the 1929 report of A. S. T. M. Committee C-9, Sub-Committee XIII on Curing, as follows:

" For the purpose of clarifying the situation the sub-committee offers the following as to its understanding of what is meant by the term ' Curing ' of Concrete and the results that may be expected from lack of ' Curing.'

" By good ' Curing ' of concrete is meant the setting up of favorable conditions for chemical action during the setting and early hardening period.

" The general opinion at the present time is that the most favorable conditions exist when the concrete is warm and moist throughout. Temperatures that are regarded as favorable in practice, range from 70° F. to somewhat above 100° F.

" Our present knowledge of the behavior of portland cement and concrete while setting and hardening indicate that if these conditions of temperature and moisture are met:

" (a) A maximum amount of cement will be hydrated; and

" (b) The concrete will be subjected to a minimum amount of volume change during the period of low strength.

"There is also evidence that when improper or inadequate curing conditions prevail, the insufficient hydration of the cement will be revealed through low strength of the concrete and lack of durability with the passage of time, and that excessive cracking will result from the volume changes due to variations in temperature and in moisture content."

PROPERTIES INDICATING EFFECTIVENESS OF CURING

This Special Committee on Curing is of the opinion that information on the following properties of concrete will indicate the relative merits of various curing methods:

(1) Moisture retention during curing. There is evidence to indicate that the amount of mixing water retained in the concrete during the curing period is an important index of the effectiveness of the various curing methods.
(2) Flexural and compressive strength.
(3) Volume changes at successive intervals.
  (a) Spacing, width and rate of formation of cracks.
  (b) Accurate measurements of changes of length.
(4) Surface condition.
  (a) Scaling or surface disintegration.
  (b) Resistance to wear.
(5) Permeability and absorption.
(6) Resistance to natural destructive agencies.
  (a) Physical such as freezing and thawing.
  (b) Chemical such as sulphate waters.

FACTORS OTHER THAN CURING WHICH AFFECT PROPERTIES OF CONCRETE

In carrying out curing studies the following factors which also affect the test results must be controlled:

(1) Uniformity of test specimens affected by such factors as
  (a) Materials, proportions, manipulation and subgrade.
  (b) Moisture and temperature condition of specimen at time of test.
(2) Testing technique. In order to avoid variations in methods it is recommended that American Society for Testing Materials procedure be followed whenever applicable.

Existing Data

Summaries of the experimental data from 36 projects are shown in Tables I to V. These investigations were mostly carried out by independent agencies under various conditions. The results were frequently contradictory due probably to other variables than the curing conditions under consideration. The committee has been much impressed with the necessity for coordinated research, rather than sporadic individual efforts. Enough effort and money have been expended for curing investigations so that the problem should be almost, if not entirely, solved. This, however, is far from being the case. Factors such as moisture content of the specimen at time of test were frequently not reported, and yet the variations in strength of the specimens due to moisture were probably greater than any differences due to curing. Certain tendencies, however, may be noted from a study of the data obtained on strength, volume change and surface condition.

EFFECT ON STRENGTH

Table I summarizes the results of tests on beams and compression specimens made in the field and in the laboratory. The values in the table are ratios, expressed as percentages, of the strengths obtained for a given method of curing to the strength of similar specimens cured with wet earth or straw. For purpose of comparison the average values for tests at all ages and for 28-day tests only are included. In order to show the range in results high, low and grand average values are reported. In these tests, calcium chloride, Hunt Process (cut back asphalt) and Curcrete (asphalt emulsion) surface treatments generally show a small range in results and gave strength 90 per cent or more of the strength of concrete cured with wet earth or straw.

In these tests sodium silicate surface treatment and calcium chloride used integrally showed strengths both above and below 90 per cent of the strength of concrete cured with wet earth or straw. Some relatively low strengths appear, particularly in the laboratory tests, which are in general lower than the field tests. Further investigations are needed to disclose the conditions under which these variations in strength occur.

Attention is called to the fact that for some of the newer methods of curing, particularly the bituminous surface treatments, only a few results were available.

TABLE I

Summary of Results of 36 Curing Investigations

(Values in Parentheses Are the Number of Projects on Which Results for a Given Method of Curing Are Based)

Method of curing	Ratios of strengths for various methods of curing to strengths for wet earth or straw curing—per cent											
	Flexure tests						Compression tests					
	Field specimens			Laboratory specimens			Field specimens			Laboratory specimens		
	High value	Low value	Grand av.	High value	Low value	Grand av.	High value	Low value	Grand av.	High value	Low value	Grand
Average of Tests at Ages up to 1 Year (Average Age Less Than 28 Days)												
No curing	94	75	84(8)	87	69	81(7)	76	76	...(1)	81	66	73(4)
Calcium chloride surface	106	92	97(9)	117	95	102(4)	94	92	93(4)	..	..	..
Calcium chloride admixture	110	74	94(11)	84	66	75(5)	134	84	103(16)	97	68	89(5)
Sodium silicate (24 h.)	101	75	87(9)	99	58	79(4)	116	86	96(11)	93	58	79(4)
Hunt process	97	92	95(2)	97	97	97(1)	89	89	..(1)	99	99	...(1)
Curcrete	104	96	99(3)	93	93	93(1)	111	97	106(4)	90	90	..(1)
Misc. bituminous	98	89	..(2)	94	86	..(2)	...	..	...	98	98	..(1)
Average of 28-Day Tests												
No curing	93	72	83(7)	87	69	80(6)	73	73	...(1)	76	62	71(3)
Calcium chloride surface	109	88	98(9)	98	94	96(3)	93	93	...(1)	..	..	..
Calcium chloride admixture	108	71	91(8)	87	66	76(5)	113	91	106(5)	103	68	90(4)
Sodium silicate	98	72	87(9)	97	58	78(4)	96	90	92(2)	93	58	77(4)
Hunt process	93	88	90(2)	90	90	90(1)	90	90	...(1)	102	102	..(1)
Curcrete	104	95	98(3)	88	88	88(1)	109	100	107(2)	89	89	..(1)
Misc. bituminous	98	86	..(2)	94	86	..(2)	...	..	...	98	98	..(1)

TABLE II

Summary of Flexural Tests at Various Ages Averaged

RATIOS OF STRENGTH TESTS FOR VARIOUS CURING METHODS TO THE STRENGTHS OF CONCRETES CURED WITH WET EARTH OR STRAW

Source of data	Cured with wet earth or straw	Ages of tests averaged	Strength by tests, expressed as a percentage of the strength of corresponding concrete cured with wet earth or straw						
Number	Days	Days	No curing	Hunt process	Curcrete	Misc. bituminous	Sodium silicate 24 hrs.	Calcium chloride admix.	Calcium chloride surface
Field Tests									
1	9	7 — 28 — 90	78.8	92.3	97.0	...	88.4	90.4	95.7
2	9	4 — 7 — 28	...	...	...	...	92.0	92.7	94.3
10	10	10 — 14 — 90 — 360	...	...	...	...	...	96.1	...
3A	7 & 28	7 — 28	82.0	...	...	(c)89.0	82.0	...	97.5
3B	10	21 — 28 — 91	...	...	...	...	(b)91.6	...	91.8
4A	2 & 6	7 — 28 — 90	94.0	...	...	...	80.0	93.0	94.0
5	10	28	...	...	103.8	...	...	94.1	...
6	10	7 — 14 — 28	...	...	...	...	...	104.1	105.8
12	10	4 — 7	83.1	...	...	...	...	110.2	...
7A	14	28	84.7	...	...	(a)97.7	...	...	102.7
8	7	4 — 8 — 15 — 29	...	...	...	...	101.2	102.8	...
10D	10	7 — 14 — 90	...	...	...	...	...	89.8	...
9C	7 & 14	7 — 14 — 28 — 90	75.2	94.6	99.0	...	75.0	...	92.3
16	14	3–7 — 14 — 28–90 — 360	82.0	...	...	...	83.0	74.0	...
17	10	3 — 7 — 14 — 28	91.0	97.0	96.1	...	93.8	82.6	99.5
Average	..	.. ..	83.8	94.6	99.0	...	87.4	93.6	97.0
Laboratory Tests									
4B	3 & 10	7 — 14 — 21 — 28	86.5	...	...	...	...	79.0	96.6
7B	10	28	85.6	...	...	...	...	...	95.1
7C	14	28	87.0	...	...	(a)94.5	...	...	98.4
9A	13	14 — 28 — 90	74.5	97.0	93.1	...	78.1	72.1	...
10C	28	28	...	...	...	(d)86.3	80.4	75.1	...
11	10	7 — 28 — 90	86.2	...	...	...	99.0	84.0	116.7
10A	28	28	69.2	...	...	...	58.3	66.0	...
Average	..	.. ..	81.5	97.0	93.1	...	79.0	75.2	101.7

Note.—(a) Blown Asphalt. (b) Applied at 7 hours. (c) Av. of a thick and thin asphalt. (d) Tar.

TABLE III

SUMMARY OF FLEXURAL TESTS, AGE 28 DAYS

RATIOS OF STRENGTH TESTS FOR VARIOUS CURING METHODS TO THE STRENGTHS OF CONCRETES CURED WITH WET EARTH OR STRAW

Source of data	Cured with wet earth or straw	Strength by tests expressed as a percentage of the strength of corresponding concrete cured with wet earth or straw						
Number	Days	No curing	Hunt process	Curcrete	Misc. bituminous	Sodium silicate 24 hours	Calcium chloride admix.	Calcium chloride surface
Field Tests								
1	9	78.0	92.9	96.6	...	88.9	89.3	97.2
2	9	...	...	...	...	92.0	96.0	96.0
3A	7 & 28	88.0	...	...	(c)86.5	85.0	...	95.0
3B	10	...	...	...	...	(b)91.7	...	94.7
4A	2 & 6	93.0	...	...	...	90.0	90.0	99.0
5	10	...	...	103.8	...	...	94.1	...
6	10	...	...	...	...	...	107.6	109.0
7A	14	84.7	...	...	(a)97.7	...	...	102 7
8	7	...	...	...	...	97.6	90.4	...
9C	7 & 14	72.0	...	...	...	74.0	...	88.0
16	14	74.0	...	...	...	72.0	71.0	...
17	10	92.9	87.5	95.1	...	95.5	92.6	96.7
Average	..	83.2	90.2	98.5	...	87.4	91.4	97.6
Laboratory Tests								
4B	3 & 10	81.5	...	...	...	...	86.2	93.6
7B	10	85.6	...	...	...	...	...	95.1
7C	14	87.0	...	...	(a)94.5	...	...	98.4
9A	13	73.1	89.8	88.5	...	74.4	67.9	...
10C	28	...	...	...	(d)86.3	80.4	75.1	...
11	10	85.4	...	...	...	97.4	87.0	...
10A	28	69.2	...	...	...	58.3	66.0	...
Average	..	80.3	89.8	88.5	...	77.6	76.4	95.7

NOTE.—(a) Blown asphalt. (b) Sodium silicate applied at 7 hours. (c) Average of thick and thin asphalt. (d) Tar.

TABLE IV

SUMMARY OF COMPRESSIVE TESTS—VARIOUS AGES AVERAGED

RATIOS OF STRENGTH TESTS OF VARIOUS CURING METHODS TO THE STRENGTHS OF CONCRETES CURED WITH EARTH OR STRAW

Source of data	Cured with wet earth or straw	Ages of tests averaged	Type of specimen	Strength by tests expressed as a percentage of the strength of corresponding concrete cured with wet earth or straw						
Number	Days	Days		No curing	Hunt process	Curcrete	Misc. bituminous	Sodium silicate 24 hr.	Calcium chloride admix.	Calcium chloride surface
			Field Tests							
1	9	6 — 29 — 90	Beam ends	75.9	88.8	97.1	...	87.9	98.8	94.0
2	9	7 — 28	6 by 12 cyl.	...	...	...	...	85.5	96.0	...
2	9	90	Cores	...	...	...	...	87.0	84.0	92.0
10	10	10 — 14 — 90 — 360	6 by 12 cyl.	...	...	...	...	...	97.5	...
10D	10	7 — 14 — 28 — 90	6 by 12 cyl.	...	...	...	...	...	95.8	...
5	10	28	Beam ends	...	...	107.6	...	...	120.9	...
5	10	28	6 by 12 cyl.	...	...	107.2	...	...	117.6	...
5	10	28	Cores	...	...	111.3	...	...	101.4	...
13	10	75 to 130	Cores	...	...	...	...	115.7	117.5	...
13	10	120	Cores	...	...	...	...	...	...	93.4
14	10	210	Cores	...	...	...	...	95.9	...	93.9
6	10	7 — 14	6 by 12 cyl.	...	...	...	...	...	116.4	...
8	7	28 — 120 — 180 — 360	Cores—1	...	...	...	...	92.5	90.2	...
8	7	180 — 360	Cores—2	...	...	...	...	108.5	103.4	...
8	7	7 — 14 — 28	Cyl.—1	...	...	...	...	98.7	97.3	...
8	7	8 — 15 — 28	Cyl.—2	...	...	...	...	98.1	134.0	...
8	7	28 — 90 — 360	Cores	...	...	...	...	101.5	92.7	...
16	14	360	Cores	...	...	...	...	89.0	89.0	...
Average	..	... .		75.9	88.8	105.8	...	96.4	103.3	93.3
			Laboratory Tests							
11	10	7 — 28 — 90	6 by 12 cyl.	80.5	...	...	...	91.9	97.0	...
9A	13	7 — 14 — 28 — 90	6 by 12 cyl.	66.3	99.2	89.7	...	73.2	77.3	...
10A	28	28	6 by 12 cyl.	74.9	...	...	...	57.6	67.6	...
10B	(a)	2 — 7-28 — 90 — 360	6 by 12 cyl.	...	...	...	...	...	109.0	...
10C	28	260	6 by 12 cyl.	...	...	...	(b)98.2	92.9	93.7	...
9B	3	120	6 by 12 cyl.	71.9	...	...	...	...	...	...
Average	..	... .		73.4	99.2	89.7	...	78.9	88.9	...

NOTE.—(a) Stored in Wet Sand until Tested. (b) Tar.

TABLE V

Summary of Compressive Tests—Age 28 Days

RATIOS OF STRENGTH TESTS OF VARIOUS CURING METHODS TO THE STRENGTHS OF CONCRETES CURED WITH WET EARTH OR STRAW

Source of data	Cured with wet earth or straw	Type of specimen	Strength by tests expressed as a percentage of the strength of corresponding concrete cured with wet earth or straw						
Number	Days		No curing	Hunt process	Curcrete	Misc. bituminous	Sodium silicate 24 hours	Calcium chloride admix.	Calcium chloride surface
			Field Tests						
1	9	Beam ends	72.7	90.3	99.9	...	90.4	101.3	93.0
2	9	6 by 12 cyl.	...	...	...	...	...	97.0	...
10D	10	6 by 12 cyl.	...	...	...	...	...	91.1	...
5	10	Beam ends	...	...	107.6	...	...	120.9	...
5	10	6 by 12 cyl.	...	...	107.2	...	...	117.6	...
5	10	Cores	...	...	113.3	...	...	101.4	...
8	7	Cores—Sec. 1	...	...	...	...	88.9	81.4	...
8	7	Cyl.—Sec. 1	...	...	...	...	97.1	106.4	...
8	7	Cyl.—Sec. 2	...	...	...	...	101.0	156.8	...
8	7	Pav. Cores	...	...	...	...	97.6	90.4	...
Average	..		72.7	90.3	107.0	...	92.0	106.4	93.0
			Laboratory Tests						
11	10	6 by 12 cyl.	75.5	...	...	...	88.8	103.0	...
9A	13	6 by 12 cyl.	61.7	102.1	89.4	...	68.1	70.2	...
10A	28	6 by 12 cyl.	74.9	...	...	...	57.6	67.6	...
10B	(a)	6 by 12 cyl.	...	...	...	...	...	114.0	...
10C	28	6 by 12 cyl.	...	...	...	(b)98.2	92.9	93.7	...
Average	..		70.7	102.1	89.4	...	76.9	89.7	...

Note.—(a) Stored in wet sand until tested. (b) Tar.

EFFECT ON VOLUME CHANGE

Reports indicate that volume changes, due to temperature and moisture content were affected by curing processes. These are ordinarily shown in pavements by the spacing of contraction cracks. In order to obtain information on this important subject, a comprehensive field survey of pavements subjected to a wide range of climatic conditions will be made. In addition certain projects are under way and others contemplated to obtain accurate measurements of changes of length of concrete cured by different methods.

EFFECT ON SURFACE CONDITION

Little definite information on the relation between curing methods and surface condition has been available. Reports from various localities indicated the need of a study of the possible relation between surface scaling and the use of a surface application of calcium chloride. As a result an extensive survey has been recently made and although there has not been sufficient time to assemble and study the data accumulated, a brief preliminary report is included herein.

Current Research Projects

Extensive research projects in curing have been recently carried out by the California, Iowa, Kansas, Tennessee and Wisconsin highway departments, the Portland Cement Association and Ohio State University. The data will be available for study the coming year. The methods outlined by this committee last June were used in some of these researches. The following research agencies have indicated their intention to conduct investigations as recommended by this committee during the coming year: Iowa, Missouri, Rhode Island, Tennessee and Wisconsin highway departments; the Portland Cement Association and Texas State College.

The "Outlines for Curing Investigations" recommended by the committee are included with this report. Careful coordinated researches in many sections of the United States are necessary for the solution of the curing problems.

SURVEY OF SURFACE CONDITION OF PAVEMENTS CURED WITH SURFACE APPLICATION OF CALCIUM CHLORIDE AND OF PAVEMENTS CURED WITH EARTH AND WATER

The Curing Committee at its meeting in June, '29, recommended that a survey to ascertain the relation between surface application of calcium chloride and surface scaling be made. In view of the fact that a large mileage of concrete pavement cured in this way is available for study, it was thought that a detailed survey of a large mileage of these pavements and of corresponding pavements cured with earth and water would develop the true facts of the case.

The Field Survey was started on August 23d, and completed on November 20th. Approximately 2000 miles of pavement was observed in the states of Minnesota, Wisconsin, Missouri and Illinois. One hundred samples of scale and 96 pictures of the scaled pavement were obtained. Due to the limited time available this report contains only a general statement of the problem and the unanalyzed findings. The projects to be surveyed were selected at random. This in most cases was an arbitrary selection by the investigator, who knew nothing of the surface condition of the pavements. The State Highway Department in each state generously furnished means of transportation and an engineer to act as an additional observer.

The observational data on each project were obtained while traversing the pavement at a speed of 10 to 15 miles per hour. These data included the speedometer reading at the start of project and at the places where scale occurred. The area of scale at each place was also determined by estimation. This method of determining the area was used because the actual measurement of the areas, due to their unsymmetrical trends would be laborious and slow, and it was felt that the results by estimation were sufficiently accurate.

The occurrence of the scale in respect to the pavement surface, location and other special construction features was also noted.

The taking of the pictures and obtaining the samples of scale completed this phase of the survey.

All of the above observational data were recorded on a special form.

The construction data, materials record and quality tests of the different projects were obtained and recorded on another special form. It is believed that the analysis and interpretation of these data in relation to the extent of scaling will reveal interesting information of sufficient quantity to permit the derivation of sound conclusions.

The typical disintegrated concrete scale was also omitted from this survey, although a few samples were taken. This scale is generally in excess of .25 inch in thickness.

From Table VI it is noted that the average thickness of the scale is approximately the same for wet earth and calcium chloride surface application method of curing.

The Location of Scale on Pavement Surface

By far the most scale appeared in a strip two to three feet wide along the outer edge of the pavement. The area adjacent to the center line showed the next greatest tendency to scale and the oil covered lane down the center of each half of the pavement showed the least. These conditions were especially true if the scaling was not extensive. In the latter case the whole surface was affected. The lack of scale in the oil covered area may be attributed to the resistance to moisture absorption by this area and the absence of abrasion by traffic.

Mechanics of Scaling

To demonstrate the true action or behavior of scaling will require a reproduction of the phenomena under direct and controlled methods. From field observations it appears that its occurrence is a function of the variation in the density of the surface layer and the concrete immediately underneath. The possible conditions that may effect this density and on which the accumulated data will undoubtedly give valuable information are: (1) Harsh mix; (2) Over finishing; (3) Too much mixing; (4) Fine and silty sand; (5) Frost action and (6) Misapplication in the use of the curing medium.

Table VI shows the general extent and nature of the surface scale found by this survey.

Expressing the extent of scaling by percentage of total area, although furnishing a suitable index of the general action of the respective curing method on the pavement surface, does not give an entirely true picture of conditions, because several of the projects have no surface defects, regardless of the curing method, while other projects show considerable scaling. In the final analysis of this survey it is hoped to show this phase by grouping the projects on a scale percentage basis.

TABLE VI

Wet Earth Curing

State	Miles surveyed	Total sq. yds. surveyed	Total sq. yds. of scale	Per cent by area	Average thickness of scale samples (Inches)
Minnesota	90.20	953,780	886.0	0.093	0.120
Wisconsin	88.00	952,138	2,149.0	0.226	0.112
Missouri	102.10	1,034,880	172.0	0.017	0.110
Illinois	297.98	3,099,009	4,557.6	0.147	0.198
Total or Average.	578.28	6,039,807	7,764.6	0.129	0.163

Calcium Chloride Surface Curing

State	Miles surveyed	Total sq. yds. surveyed	Total sq. yds. of scale	Per cent by area	Average thickness of scale samples (Inches)
Minnesota	114.40	1,270,248	4,937.0	0.389	0.150
Wisconsin	129.00	1,481,094	3,184.0	0.215	0.118
Missouri	260.20	2,749,444	1,929.0	0.070	0.139
Illinois	605.78	6,402,222	10,835.1	0.168	0.165
Total or Average.	1,109.38	11,903,008	20,885.1	0.175	0.152

Average age of wet earth cured concrete pavements—8 yrs.
Average age of Calcium Chloride cured pavements—5 yrs.
NOTE.—There is an additional 320 miles to be compiled and added to this table.

CLASSIFICATION OF SCALE

The survey included only the type of scaling which was of an appreciable depth. The thin surface peel, commonly called "Paper Scale," was not classed as a scale. This "Paper Scale" is always less than .05 per cent inch in thickness and is unnoticeable except when driving at a low rate of speed and with a favorable angle of light reflection from the pavement.

OUTLINE FOR SUGGESTED CURING INVESTIGATIONS

This outline is based on the assumption that there are five distinct qualities to which a curing agent should be subjected before a final value is placed on its effectiveness. These are: (1) Flexural and compressive strength, (2) Volume change, (3) Surface condition, (4) Permeability, and (5) Durability as revealed through freezing and thawing.

The field tests recommended are for the purpose of clearing up the situation as respects the characteristics of the various methods in vogue at the present time, in so far as possible.

The laboratory tests proposed are a start toward developing the fundamental principles upon which proper curing depends.

FIELD TESTS

(1) *General Proposal.* The project contemplates investigation of several methods of curing of which one would be a good curing method to be used as a standard for comparison and one would be a minimum curing method to furnish information as to the extent of curing offered by the natural conditions prevailing during the tests. It is assumed that several other methods will be investigated progressively on the same project. It is assumed that the unit for each method of curing will be the length of pavement to be placed in one day's operation, say about 1000 feet. Each change of curing method should take place at noon and the same method of curing then continued until the following noon. This constitutes one unit for comparison.

The following alternation of curing methods is suggested:

(1) Minimum curing unit.
(2) Standard unit.
(3) Experimental unit No. 1.
(4) Standard unit.
(5) Experimental unit No. 2.
(6) Standard unit.
(7) Experimental unit No. 3.

If more than three experimental methods are used the units involving them should be alternated with standard unit as above, finishing with the last experimental unit.

The operation up to this stage will constitute one round of tests. This round should be followed by at least one more round carried out in the same manner. More should be added if possible.

The following outline gives recommended details regarding the procedure:

(2) *Standard of Comparison.* Wet burlap for first 24 hours, then 2 inches of moist earth for nine days. Require report to indicate frequency of wetting and approximate quantity of water.

(3) *Minimum Curing.* Wet burlap for first 24 hours. See that no accidental curing occurs by leakage from hose or other preventable sources.

(4) *Experimental Methods.* These optional with experimental agency. Suggested methods:

(a) Curcrete, Hunt process and any other bituminous coatings.
(b) Calcium chloride, surface application.
(c) Sodium silicate, surface application.
(d) Calcium chloride admixture (2 lbs. per bag of cement).

(5) *Suggested Procedure for Experimental Methods.* ((a) to (d) above.)

(a) Spray bituminous coatings on surface in amounts recommended by manufacturers as soon as practicable after finishing operations.
(b) Cover with wet burlap 24 hours and follow immediately with application of 2 lbs. calcium chloride per square yard.
(c) Cover with wet burlap 24 hours and follow immediately with application of sodium silicate (soda-silicate ratio 1 to 3.25) solution approximately 37 degrees *Be′* sprayed or broomed over surface using 1.1 lbs. of the solution per square yard of surface.
(d) Use admixture of 2 lbs.* calcium chloride per sack of cement dissolved in mixing water. Cover with wet burlap first 24 hours.

(6) *Data to be Secured and Reported:*

(a) Strength of cores at ages of 28 and 90 days and 1 year; at other ages if desired. The cores should preferably be 5½ inches in diameter. Each round of cores should be taken so as to give one specimen for each age, all from a single batch. Six rounds, one from each of six different batches from each section will constitute a set. Three of the batches selected should be from those placed in the forenoon and three in the afternoon.

The cores should preferably be drilled not earlier than 5 days prior to the date on which they are due to be tested. It is recommended that for transporting the cores be wrapped in paper.

It is recommended that the ends of the cores be made plane and parallel by grinding. If this is not practicable, capping with baselite (see *Engineering News-Record,*

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* Under some conditions it may be desirable to use less than 2 lbs. to give proper finishing.

Nov. 22, 1928, p. 777) or other satisfactory materials recommended.

Caps should be plane and parallel. They should be as thin as possible, especially if the capping material is not more rigid than the concrete of the core. All cores shall be soaked prior to testing. The following periods of soaking are recommended: 28 day cores, 24 hr.; 90 day cores, 48 hr.; 1 year cores, 96 hr.

Specimens should be weighed immediately before and after soaking to determine amount of absorption.

(b) Complete test reports on all materials including cement.
(c) Proportions (absolute volume) and method of measuring.
(d) Time of mix and type of mixer.
(e) Amount of water and method of control.
(f) Subgrade:
    (1) Character: Detail survey for each section.
    (2) Treatment.
(g) Placement and finishing methods.
    (1) Type of finishing machine.
    (2) Number of times finishing machine operation was repeated.
    (3) Report in detail all manipulation of surface.
    (4) Character of surface after finishing, dry, wet, glistening, soupy, etc.
(h) Elapsed time between finishing concrete and placing burlap.
(i) Notation and extent of cracks or other surface defects present before placing burlap or other covering: or on removal of burlap.
(j) Temperature record beginning with placement of concrete.
    (1) Air, wet and dry bulbs.
    (2) Concrete at top and bottom surfaces and at intermediate points if possible. Minimum of two locations per 1000 foot section; one in a. m. and one in p. m. Readings to be taken hourly until heat of reaction has been dissipated and three times a day until normal cycles have been established.
(k) Amount of sunshine.
(l) Rainfall. Amount, time and duration.
(m) Strain measurements on top surface. Longitudinally and transversely at a point 6 feet from outside edge of slab Ell (L) shape figure at two places in each section

adjacent to temperature stations. Take readings at same time intervals as for temperature. See attached instructions for strain gage measurements.

(n) Crack and check survey.
  (1) Examine before covering with burlap or bituminous coatings and plot all cracks giving data of appearance.
  (2) Repeat on removal of burlap and at intervals thereafter sufficiently frequent to determine characteristic rate of formation of cracks.

NOTE.—The use of a light film of oil will bring out cracks not easily detected otherwise.

(7) *Surface Condition.*
  (a) Note any surface scaling.
  (b) Wear resistance. The absence of a suitable wear machine for field use has greatly obstructed the acquiring of valuable data on this important quality of concrete in relation to curing. The Bureau of Public Roads has recently developed a machine for determining surface wear of pavements and is at present determining its practicability. A description of this machine and the test data obtained will be made through the official publication of the Bureau of Public Roads in the near future.
  (c) Moisture retained in surface. If practicable determine moisture retained in the top 2 inches or less of concrete by specimens secured from the pavement by the following method. Embed 2 ply cheese-cloth 2 inches (or less) below the surface as shown in Figure 1.

  (6) Remove the disc of concrete at the following periods, (1) immediately after placing, (2) at end of 1, 7, and 28 days. Weigh concrete and dry at 105° C. to constant weight and weigh again. Take three specimens for each section for each age.

(8) *Permeability.* This quality of the concrete cannot be determined in the field, and for that reason will be omitted. Sections of the drilled cores or the disc for moisture retained in surface may be used for this test at laboratory.

(9) *Durability as Revealed Through Alternate Freezing and Thawing.* These cycles cannot be accelerated in the field as they depend upon climatic conditions and for that reason cannot be standardized. Periodical surveys in conjunction with the number of cycles and the

temperature intensity may reveal valuable data on this phase of the concrete quality.

(10) *General Remarks.* In carrying out the field tests, it is suggested that some mutual agreement be completed with the contractor either in the form of some special clause in the original contract or by some other amicable arrangement. The inconveniences to the contractor were kept in mind throughout the outline and are believed to be minimized to the limit, in obtaining the necessary information.

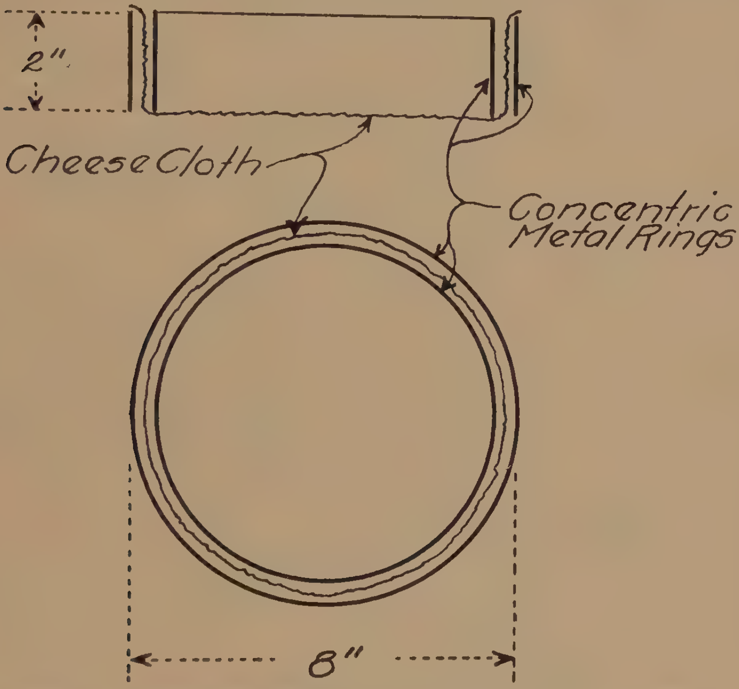

Figure 1. Method for Securing 2 by 8 Inch Disk from Pavement Surface for Moisture Tests

LABORATORY TESTS

(A) The following curing methods are suggested.
  (1) Wet sand.
  (2) No—curing.
    (a) Tested as cured.
    (b) Tested wet.
  (3) Hunt process. Two different amounts.
  (4) Curcrete. Two different amounts.
  (5) Sodium silicate surface.
  (6) Calcium chloride admixture.

- (7) Calcium chloride 1¼ lb. Admixture with hot materials.
- (8) Calcium chloride surface.
- (9) 72 hours of wet burlap (allowed to evaporate).
- (10) Materials hot. No cure.

(B) Moisture retention.
- (1) Measure moisture loss daily for seven days, then at following intervals: 14, 28, 90 and 365 days.

(C) Strength tests.
- (1) Ages of test, 3, 7, 28, 90 and 365 days.
- (2) 6 x 6 x 32 inches beams. End of beams or cores from beams desirable for compressive strength tests.
- (3) Make specimens in rounds so that one specimen for each method of curing is made on each day for five different days. These five specimens for each curing method shall constitute a set.

(D) Proportions.
- (1) Absolute volume. Materials available shall be proportioned by weight to conform to an absolute volume mix of 1 : 2.67 : 3.75. (This is equivalent to a 1 : 2 : 3 arbitrary volumetric mix in which the coarse aggregate contains 40 per cent voids and the sand 36 per cent voids and the specific gravity of both is 2.65.)
- (2) Cement. (Standard Portland.)
    - (a) Constant. (Same brand and shipment.)
    - (b) Mix the cement and store in air-tight containers.
    - (c) Standard physical tests.
        - (1) Beginning of test.
        - (2) End of test.
    - (d) One set of chemical tests.
    - (e) Must meet A. S. T. M. requirements.
- (3) Sand.
    - (a) Must meet requirements C-33-28T A. S. T. M.
    - (b) All passing ¼ mesh sieve.
- (4) Coarse Aggregate.
    - (a) Maximum size 1½ inches.
    - (b) Separate into three sizes.
- (5) Aggregates in general.
    - (a) All aggregates in stock at start of test.
    - (b) Proportion separately by dry weight.
    - (c) Use aggregates with as low absorption as possible.

- (d) Uniform composition.
- (e) Use aggregate with no surface moisture. Allow for absorption and surface mositure.

(6) Water content.
- (a) Water-cement ratio constant.
- (b) Plastic mix.
- (c) Report water-cement ratio.
- (d) Measure and report slump and flow.
- (e) Use tap water tempered to 70° F.

(7) Mixing.
- (a) Use laboratory concrete mixer (tilting mixer if possible).
- (b) Mix two (2) minutes.
- (c) Clean mixer at start with water and between batches, being careful to get water out of drum.
- (d) Dump concrete into pan and mix with shovel to avoid segregation.

(8) Beam construction.
- (a) Use tin lined forms (details of construction will be available).
- (b) Place concrete in one layer and spade sides. Strike off top with combined transverse and longitudinal motion with wooden straight edge after water sheen disappears. Finish with one application of wood float. (Caution against surface laitance.)

(9) Treatment of beams after construction for the various curing methods.

Standard.
- (a) Cover with wet burlap. Test one set of beams in damp condition after continious curing in moist room.
- (b) Cover two sets of beams with wet burlap and cover burlap with canvas on some water-proof paper for 12 to 24 hours. Place 2 inches or more of wet sand and keep saturated for 9 days. Test one set immediately after removing covering up to 7 days and the remainder of beams in this set to be tested in condition which exists when due to be tested. The second set of beams to be

soaked for periods indicated under no curing prior to testing.

No Curing. Use no wet burlap. Test one set dry or in cured condition and immerse the other set in water after removing forms (tin) for:

Period of test	Duration of immersion
3 day	6 hours
7 day	24 hours
28 day	48 hours
90 day	72 hours
365 day	7 days

Hunt process. No wet burlap. Apply bituminous covering immediately after water sheen disappears. On one set of beams use amount of material recommended by manufactures and on the other set use 50 per cent additional to that recommended. Apply with suitable spray using extreme caution to see that covering is properly and uniformly applied. Must have absolute record of amount put on. Also carefully observe surface with large reading glass for pin holes in bituminous coatings.

Curcrete. Same as Hunt.

Sodium silicate. The method of application, ratio of sodium to silicate, concentration and the time of application to conform with recommendation of manufacturers. Use wet burlap for first 24 hours if application is on following day.

Calcium chloride. Admixture, 2 pounds per bag of cement dissolved in mixing water. Surface, 2 pounds per sq. yard. In both cases use wet burlap for just 24 hours. (Use manufacturer's recommendations.)

GENERAL REMARKS

After specimens have set in tin lined forms for 24 hours, place a heavy asphalt or melted paraffin around edges of all the beams in such a manner that the crack between the tin form and the concrete specimen due to shrinkage is completely sealed. This can be accomplished by placing the asphalt (or paraffin) in a small ridge, so that about ¼ inch of the edges of the concrete is coated and extending over the crack and adhering to the vertical outside face of the tin for a ¼ or ½ inch below the top edge. This will prevent moisture losses from other than the cured surface. Prior to testing, beams to be soaked in water for periods indicated under no curing.

(E) Surface condition.

Abrasion tests are desirable, but methods are not sufficiently developed. Laboratory may develop method suitable for this test.

The following methods are used:

(1) Jones—Talbot Rattler Test (Bulletin 10 Structural Materials Research Laboratory).
(2) Ball Indentation Test. (Bulletin No. 12. Engineering Experiment Station, Purdue University.)
(3) Dory Hardness machine and modifications of this test. (Dory hardness machine standard equipment in most highway laboratories.)
(4) National Crushed Stone Association Method.
(5) Bureau of public roads field machine for wear. (To be reported soon in Public Roads.)

(F) Shrinkage.

(1) Strain gage measurements.

Strain Gage Measurements

The type of strain gage to be used probably will depend largely upon what is most accessible. The Berry Strain Gage can be used very satisfactorily. Somewhat better results have been obtained with the Whittemore gage of the type used on the Stevenson Creek Dam. (Manufactured by H. L Whittemore Bureau of Standards, Washington, D. C.) A 10-inch gage length has been found to give as small error in inches per inch as a 20-inch gage length.

The Berry gage is put out in 8-inch and 20-inch gage lengths. Probably of these the 20-inch gage length would be best. With the Whittemore gage, probably 10-inch would be best.

For using the strain gage, metal plugs made by cutting 1-inch lengths of ½-inch round bars may be set in the concrete at the proper distance apart with one end of the plug flush with the surface of the concrete, or slightly below the surface for protection of the plug. For the long-time tests it is desirable that a non-corrosive metal such as stainless steel or monel metal be used for the plugs.

A No. 54 drill is suitable for drilling the gage holes. The use of a special drill with the twist portion only about ⅛-inch long helps greatly in obviating breakage of drills. The holes may be drilled either before or after placing the plugs. Probably drilling after placing will generally be the most satisfactory. The holes should not be countersunk, but removal of burr from the edge of the hole is desirable. This may be done by inserting the point of a center punch in the hole and turning the punch at the same time that light pressure is applied by hand. The best finishing of the holes is that which comes from continued use of the strain gage point in them. Therefore, there is great value from the standpoint of accuracy in taking a number of sets of zero readings.

To guard against accidental changes in the strain gage, readings should be taken on standard reference bar of constant length at intervals not less frequent than at the beginning and end of each series of readings. It should not be difficult to secure invar steel with a coefficient of expansion as low as 0.0000012 inches per inch per degree F. (this is about $\frac{1}{5}$ the coefficient of ordinary steel), though the coefficients differ with different lots of invar. Such steel as this should be satisfactory for use as a standard reference bar. However, an ordinary steel bar kept in ice would also serve as a satisfactory reference bar but this is usually more troublesome than to secure invar bar.

Usually strain gages are made of invar steel, but if one is used which is made of ordinary steel more complete dependence upon a standard reference bar is necessary and in this case readings on the reference bar should be taken at intervals of not more than 10 minutes at the beginning of a series of readings when the instrument is being warmed up by contact with the hands, and not more than 20 minutes after it has been warmed up. Great care must also be used to keep the hands off the side bars of a plain (not invar) steel instrument and to handle it only at the ends. It is well, but not essential, to observe the same precaution in handling an invar steel gage. Felt coverings have some times been applied to strain gages to protect them from temperature effects.

The subject of strain gage operation has been discussed at length in a paper, " The Use of the Strain Gage in the Testing of Materials " by W. A. Slater and H. F. Moore, Proceedings, American Society for Testing Materials, 1913, p. 1019, and in Bulletin 64, University of Illinois Engineering Experiment Station, " Testing of Reinforced Concrete Buildings under Load," by A. N. Talbot and W. A. Slater. Much improvement in strain gages has been made since these papers were written, but this does not affect greatly the discussion of use except as to the degree of dependence upon readings on a standard bar.

REPORT OF THE COMMITTEE ON THE CAUSES AND PREVENTION OF HIGHWAY ACCIDENTS

KNIGHT DUNLAP, *Chairman*

Professor of Experimental Psychology, Johns Hopkins University

This committee has been delayed in getting under way this fall, by causes which were unavoidable, and which need not be detailed here. There has been criticism of the committee on various grounds, and there has been doubt and confusion concerning the purposes of the committee, not only by those outside, but also on the part of the committee itself. The chairman, therefore, felt it necessary to spend some time in collecting information from members of the committee and from a number of other persons who had been concerned in the original organization of the committee, and from others whose interests are closely allied to those the committee might be presumed to have. A meeting of the committee was then held, on November 16, 1929, in Washington, and the situation was presented to it. The committee considered the situation carefully, and arrived at definite and unanimous decisions on the following points:

1. It seemed clear that there is a field for work for this committee, without overlapping with other agencies engaged in research on problems of traffic and transportation, but in cooperation with them.
2. It was agreed that it would be relatively unproductive to separate the engineering problems from the psychological problems.
3. It was definitely voted that the committee should be enlarged to include non-representational members skilled in educational and psychological problems.
4. It was agreed that the committee should serve in the main in an advisory and administrative capacity, delegating active work to appropriately constituted sub-committees.
5. It was decided to proceed at once with the formation of a research program, adequately detailed as to precise problems and personnel requirements, and including estimates of costs.
6. It was the general sense of the meeting that although the committee should aid in every suitable way in the improvement of statistical data concerning accidents, it is unwise for the committee to engage in the work of collecting statistical data of this sort.

7. It was agreed that the program should include problems of whatever type may offer reasonable assurance of results applicable to the increase of safety of motor traffic and transportation, when such research shall be found otherwise not provided for; and that such research should not wait upon the determining of the question of whether the danger factors at the basis of the proposed research are major or minor factors, if they are known to be actual factors conducive or contributory to accidents.
8. After lengthy discussion of various danger factors concerning which facts essential for the improvement or remedy of existing conditions are not fully known, it was decided to form a sub-committee to draft a provisional research program for submission to the general committee. The sub-committee as appointed consists of Mr. Eynon, Dr. Dickinson, and Mr. Eliot, and is engaged in its duties.

The committee does not expect to make startlingly rapid progress. It does expect to elaborate this winter an adequate research program in which every item will have been well considered and planned from a practical point of view. It has further committed itself to the active carrying out of the research, if and when, adequate funds are obtained. The committee does not expect that its members, will actually do the research. We cannot expect the organizations with which the members are identified to donate more than the time and energy required for the general planning and supervision. The actual research must be carried out by men selected for their specific abilities and experience, and paid for their services, or must be assigned to agencies already so organized or constituted as adequately to conduct the particular investigations so assigned. It is deemed probable that both methods must be employed, each definite problem or group of problems being entrusted to the research personnel best fitted to undertake it, and in the place where it can adequately be solved.

The entire program undertaken, however, should be integratively supervised and fostered by the committee. It is possible such supervision would best be exercised through a director responsible to and advised by the committee and its sub-committees.

No request for, or further suggestion of, financial support will be made until the committee is able to present a plan which shall clearly merit strong support.

I may add that in the opinion of its chairman, the committee is proceeding in exactly the proper direction, and is making progress with satisfactory rapidity. The problem planning will require a considerable expenditure of time by men upon whom other demands are heavy. It is true that the chairman of the committee ought to give more time and energy to this work than is possible for the present chairman, hence he hopes a more active chairman may be shortly found.

APPENDIX

TO REPORT OF THE COMMITTEE ON CAUSES AND PREVENTION OF ACCIDENTS

A Partial List of Topics on Which Definite Research is Needed on One or More Problems Directly Related to Safety in Motor Traffic and Transportation

1. The design and construction of road beds and other highway features is, of course, a prime factor in safety. It is probable that the Committee on Causes and Prevention of Accidents may be able usefully to cooperate with the other committees of the Highway Research Board on certain problems in this field.
2. Motor equipment, including brakes, headlights, horns, etc.
3. Signs and signals for non-urban use; including problems of shape, size, color, and lettering; avoidance or minimizing of reading requirement; and placement to minimize congestion, erratic driving and other menaces due to the consulting of signs. (The present standardization is not final, and important factors have apparently been omitted from consideration in some cases.)
4. Urban and suburban signs and signals, including stop-go signals, boulevard stops, one-way and other directional signs, and parking regulatory signs. (The use of these signs is in a chaotic condition, detrimental to safety; and the types of sign are not optimal. *A priori* reasoning and hasty generalizations have made trouble here.)
5. Registration plates, including legibility of the numerals, effects of coloration, and the facilitation of memory by adequate and uniform systems. (This is an important factor in law enforcement and law observance. Some states have plates which would almost seem to have been designed to prevent accurate reading and retention.)
6. Pedestrian movement and control.

7. Smoother utilization of street space, and menaces due to parking.
8. Under and over regulation by law and ordinance; methods of enforcement; and the effect of these on attitudes and habits of drivers. (Applied psychology for traffic officers enters here.)
9. The licensing of drivers and chauffeurs, including the determination of useful and practicable limits of examinations and tests, and the problems of defective, diseased and psychopathic candidates. (These problems have been treated in an *a priori* fashion to too great an extent).
10. Habitually unsafe drivers, including their detection, certification, and cure or elimination.
11. Education of drivers in the art and science of controlling motor cars. (Little systematic attention has been paid to this for the bulk of the driving population, most of whom badly need education.)
12. Education of pedestrians, especially of children.

MINUTES OF BUSINESS MEETING

HIGHWAY RESEARCH BOARD

December 13, 1929

The meeting was called to order at 2:00 P. M. with Chairman F. H. Eno presiding.

Present:

- Executive Committee Members
  - F. H. Eno, Chairman
  - H. S. Mattimore, Vice-Chairman
  - William Spraragen
  - T. H. MacDonald
  - A. J. Brosseau
  - H. C. Dickinson
  - E. W. James
  - A. N. Johnson
  - G. E. Hamlin
  - C. M. Upham
- Director—R. W. Crum
- Member Organization Representatives
  - American Association of State Highway Officials
    - E. W. James
  - American Chemical Society
    - H. L. Sherman
  - American Concrete Institute
    - Duff A. Abrams
  - American Institute of Consulting Engineers
    - Philip W. Henry
  - American Road Builders' Association
    - Charles M. Upham
  - American Society of Civil Engineers
    - H. G. Shirley
  - American Society of Mechanical Engineers
    - J. G. Bergquist
  - American Society for Testing Materials
    - Prevost Hubbard
  - Asphalt Association
    - Prevost Hubbard

Association of American State Geologists
    E. F. Bean
Association of Land Grant Colleges
    Anson Marston
Association of Asphalt Paving Technologists
    Walter J. Emmons
Bureau of Public Roads
    Thos. H. MacDonald
National Automobile Chamber of Commerce
    A. J. Brosseau
National Crushed Stone Association
    A. T. Goldbeck
National Paving Brick Manufacturers Association
    G. F. Schlesinger
National Sand and Gravel Association
    Stanton Walker
Portland Cement Association
    E. M. Fleming
Society of American Military Engineers
    H. C. Whitehurst

Minutes of last meeting

Motion: That the minutes of the last business meeting be approved as published in the Eighth Proceedings.

Approved.

The following interim actions of the Executive Committee were presented:

1. Discharge of Committee No. 7 on Economic Theory of Highway Improvement.
2. Organization of Committee on Highway Transportation Costs.
3. Organization of Committee on Mineral Aggregates.
4. Contribution of $900.00 to historical research work of the American Association of State Highway Officials.
5. Revision of committee organization as recommended in the report adopted by the Executive Committee on November 29, 1929.

Motion: That the interim actions of the Executive Committee be approved.

Approved.

New members of the Board as result of letter ballot

The following list was presented by the Chairman:

Asphalt Association
    L. M. Law, Representative
    J. E. Pennybacker, Alternate

National Crushed Stone Association
    A. T. Goldbeck, Representative
    Otho M. Graves, Alternate
National Paving Brick Manufacturers Association
    G. F. Schlesinger, Representative
    Q. A. Campbell, Alternate
National Sand and Gravel Association
    Stanton Walker, Representative
    C. E. Proudley, Alternate
Portland Cement Association
    E. M. Fleming, Representative
    A. A. Anderson, Alternate

REPORT OF THE NOMINATING COMMITTEE

The Nominating Committee respectfully submits the following nominations for members of the Executive Committee to fill the vacancies now on the Board:

A. J. Brosseau, National Automobile Chamber of Commerce
H. K. Bishop, Chief, Division of Construction, U. S. Bureau of Public Roads
C. J. Tilden, Professor of Civil Engineering, Yale University.

Very truly yours,
Philip W. Henry, *Chairman,*
E. F. Bean,
G. F. Schlesinger,
H. C. Whitehurst,
H. G. Shirley,
Nominating Committee.

It was explained by the Chairman that these nominations are for the places of A. J. Brosseau, E. W. James and A. N. Johnson, whose terms have expired.

Motion: That the Director be instructed to cast one ballot for the nominees presented by the Nominating Committee.

Approved.

The Director called attention to the need for establishing a Research Information Service for the collection and dissemination of research information, which, he stated, was one of the original purposes in establishing the Board.

The Director presented a report of the financial transactions of the Board for 1929.

Motion: That the financial report of the Director be accepted.

Approved.

Adjourned at 2:45 P. M.

REGISTRATION AT NINTH ANNUAL MEETING

December 12 and 13, 1929

Aaron, Henry	U. S. Bureau of Public Roads	Washington, D. C.
Abbot, L. A.	U. S. Bureau of Public Roads	Washington, D. C.
Abercrombie, W. F.	State Highway Department	East Point, Ga.
Abrams, Duff A.	International Cement Corporation	New York City.
Adams, Walter S.	Mount Wilson Observatory	Pasadena, Calif.
Ahler, C. C.	State Highway Department	Albany, N. Y.
Albright, J. C.	Department of Public Works	Culpeper, Va.
Allen, L. L.	Department of Highways	St. Paul, Minn.
Allen, T. Warren	U. S. Bureau of Public Roads	Washington, D. C.
Anderton, B. A.	The Barrett Company	Edgewater, N. J.
Babcock, H. D.	Mount Wilson Observatory	Pasadena, Calif.
Baier, H. M.	American Tar Company	Malden, Mass.
Bair, H. J.	Toncan Culvert Mfgrs. Association	Massillon, Ohio.
Barton, Wm. H., Jr.	University of Pennsylvania	Philadelphia, Pa.
Bauer, E. E.	University of Illinois	Urbana, Ill.
Bayly, L. G.	U. S. Bureau of Public Roads	Washington, D. C.
Beadle, J. B.	Brock and Weymouth Incorporated	Philadelphia, Pa.
Bean, E. F.	Wisconsin Geological Survey	Madison, Wis.
Begg, R. B. H.	Virginia Polytechnic Institute	Blacksburg, Va.
Bell, A. H.	State Highway Department	Richmond, Va.
Benkelman, A. C.	State Highway Department	Lansing, Mich.
Bergquist, J. G.	American Society of Mechanical Engineers	New York City.
Berry, H. C.	University of Pennsylvania	Philadelphia, Pa.
Bigler, H. P.	Rail Steel Bar Association	Chicago, Ill.
Bischoff, R. E.	State Highway Department	Marietta, Ohio.
Blaum, Wm. L.	Department of Public Works	Albany, N. Y.
Bleecker, W. H.	Page Steel and Wire Company	New York City.
Boggs, I. H.	Interstate Amiesite Company	Martinsburg, W. Va.
Bosch, Frank B.	Central Construction Corporation	Harrisburg, Pa.
Boublitz, Harry D.	Student—University of Maryland	College Park, Md.
Bowman, H. L.	Drexel Institute	Philadelphia, Pa.
Boyd, J. R.	National Crushed Stone Association	Washington, D. C.
Brandt, A. W.	Department of Public Works	Albany, N. Y.
Bricker, J. G.	Flexible Road Joint Machine Company	Warren, Ohio.
Brosseau, A. J.	Mack Trucks, Incorporated	New York City.
Brown, R. D.	U. S. Bureau of Public Roads	Washington, D. C.
Bruce, A. G.	U. S. Bureau of Public Roads	Washington, D. C.
Buchanan, Jas. A.	U. S. Bureau of Public Roads	Washington, D. C.
Burch, Jas. S.	American Road Builders' Association	Washington, D. C.
Burton, Victor R.	State Highway Department	Lansing, Mich.
Butler, Jos. B.	Missouri School of Mines	Rolla, Mo.

Campbell, Q. A.	National Paving Brick Manufacturers Association	Washington, D. C.
Carmick, L. G.	U. S. Bureau of Public Roads	Washington, D. C.
Cerrito, A. F.	Student—University of Maryland	College Park, Md.
Chubb, J. H.	Pa. Dixie Cement Corporation	New York City.
Clemmer, H F.	Solvay Sales Corporation	New York City.
Cockey, J. P.	American Road Builders' Association	Washington, D. C.
Compton, R. Keith, Jr.	Testing Engineer	Richmond, Va.
Conahey, George	Johns-Manville Corporation	New York City.
Connell, Wm. H.	Regional Planning Federation	Philadelphia, Pa.
Conner, C. N.	American Road Builders' Association	Washington, D. C.
Conrad, L. E.	Kansas State College	Manhattan, Kans.
Crandell, John S.	University of Illinois	Urbana, Ill.
Cullimore, W. H.	Bureau of Highways	Baltimore, Md.
Curran, Frank B.	U. S. Department of Commerce	Washington, D. C.
Curtiss, C. D.	U. S. Bureau of Public Roads	Washington, D. C.
Darwin, W. P.	Portland Cement Association	Washington, D. C.
Davis, Fred A.	State Road Commission	Morgantown, W. Va.
Davis, H. M.	Standard Concrete Apparatus Co.	Washington, D. C.
Dawson, R. F.	University of Texas	Austin, Texas.
Devereaux, R. L.	U. S. Bureau of Public Roads	Washington, D. C.
Dickinson, H. C.	U. S. Bureau of Standards	Washington, D. C.
Donohue, Jerry	State Highway Commission	Madison, Wis.
Dreisbach, E. E.	Standard Lime and Stone Company	Martinsburg, W. Va.
Dunham, F. A.	Consulting Engineer	Plainfield, N. J.
Durant, G. J.	Froehling & Robertson, Inc.	Richmond, Va.
Edge, W. S.	National Steel Fabric Company	Pittsburgh, Pa.
Eliot, Wm. G., 3d.	U. S. Bureau of Public Roads	Washington, D. C.
Emmons, W. J.	State Highway Laboratory	Lansing, Mich.
Eno, F. H.	Ohio State University	Columbus, Ohio.
Eno, Wm. P.	Eno Foundation for Highway Traffic Regulation	Washington, D. C.
Evans, Edwin M.	Civil Engineer	Chevy Chase, Md.
Fairbank, H. S.	U. S. Bureau of Public Roads	Washington, D. C.
Fleming, E. M.	Portland Cement Association	Chicago, Ill.
Forrer, J. J.	State Highway Department	Richmond, Va.
Forrest, C. N.	The Barber Asphalt Company	Philadelphia, Pa.
Freeman, P. J.	Department of Public Works	Pittsburgh, Pa.
French, Owen B.	George Washington University	Washington, D. C.
Fullenwider, C. V. R.	The Philip Carey Company	Cincinnati, Ohio.
Gallier, John F.	State Highway Department	Columbus, Ohio.
Gemeny, A. L.	U. S. Bureau of Public Roads	Washington, D. C.
Gilpin, F. H.	The Texas Company	New York City.
Glidden, Wm. R.	Department of Highways	Richmond, Va.
Goldbeck, A. T.	National Crushed Stone Association	Washington, D. C.
Gonnerman, H. F.	Portland Cement Association	Chicago, Ill.
Gordon, A. F.	U. S. Bureau of Public Roads	Washington, D. C.
Gordon, J. M.	Student—University of Maryland	College Park, Md.

Graves, O. M.	General Crushed Stone Company	Easton, Pa.
Grubb, Chas. E.	County Engineer	Wilmington, Del.

Hagan, I. B.	State Highway Commission	Augusta, Me.
Hague, A. T.	Standard Oil Company (Indiana)	Chicago, Ill.
Hamlin, G. E.	State Highway Department	Hartford, Conn.
Hansard, O. H.	T. L. Herbert & Sons	Nashville, Tenn.
Hardenbergh, W. A.	Associate Editor—Public Works	New York City.
Hayden, H. P.	The Barber Asphalt Company	Maurer, N. J.
Heltzel, J. N.	Heltzel Steel Form & Wire Company	Warren, Ohio.
Henstreet, H. J.	U. S. Bureau of Public Roads	Troy, N. Y.
Henderson, G. H.	State Highway Department	Providence, R. I.
Henry, Philip W.	American Institute of Consulting Engineers	New York City.
Herschel, W. H.	U. S. Bureau of Standards	Washington, D. C.
Hine, H. H.	Student—University of Maryland	College Park, Md.
Hinkle, A. H.	State Highway Commission	Indianapolis, Ind.
Hinman, D. E.	American Steel & Wire Company	New York City.
Hitchcock, W. W.	Michigan State College	East Lansing, Mich.
Hogentogler, C. A.	U. S. Bureau of Public Roads	Washington, D. C.
Hubbard, Prevost	The Asphalt Association	New York City.
Hughes, H. W.	Supervisor, Division of Tests and Materials	Rochester, N. Y.
Hunt, C. B.	District Building	Washington, D. C.
Hutchinson, G. W.	Riverton Line Company	Riverton, Va.
Hvass, Chas. T.	Chas. Hvass and Company	New York City.

Jackson, F. H.	U. S. Bureau of Public Roads	Washington, D. C.
James, E. W.	U. S. Bureau of Public Roads	Washington, D. C.
Johnson, A. N.	University of Maryland	College Park, Md.
Johnson, Maro	American Railway Engineering Association	Chicago, Ill.
Johnson, Pyke	National Automobile Chamber of Commerce	Washington, D. C.
Johnston, C. M.	U. S. Bureau of Public Roads	Washington, D. C.

Kane, Daniel R.	State Highway Department	Deep River, Conn.
Keily, John V.	State Board of Public Roads	Providence, R. I.
Kellerman, Wm. F.	U. S. Bureau of Public Roads	Washington, D. C.
Kelley, E. F.	U. S. Bureau of Public Roads	Washington, D. C.
Kimball, C. S.	Portland Cement Association	Washington, D. C.
Kimber, Geo. H.	Solvay Sales Corporation	New York City.
Kipp, O. L.	Minnesota Highway Department	St. Paul, Minn.
Kintenacher, K. G.	State Highway Commission	Madison, Wis.
Kirk, H. J.	Consulting Engineer	Columbus, Ohio.
Klinkicht, W. R.	The Pollak Steel Company	Detroit, Mich.
Knauff, W. K.	State Road Commission	Keyser, W. Va.
Knopf, Samuel	State Highway Department	Dover, Del.
Kriege, H. F.	France Stone Company	Toledo, Ohio.
Krynine, D. P.	Professor of Highway Engineering	Moscow, Russia.
Kubitz, W. E.	American Steel and Wire Company	Baltimore, Md.

Lambert, C. W	State Highway Department	Richmond, Va
Lang, F. C	State Highway Department	St. Paul, Minn.
Lapham, John R	George Washington University	Washington, D. C.
Lay, W. E	University of Michigan	Ann Arbor, Mich.
Lepphard, F. E	Student—University of Maryland	College Park, Md.
Levison, A. A	Blaw Knox Company	Pittsburgh, Pa.
Ligon, V. P	State Highway Department	Frankfort, Ky.
Livingston, A. G	State Highway Department	Dover, Del.
Lloyd, M. E	Student—University of Maryland	College Park, Md.
Lloyd, M. G	U. S. Bureau of Standards	Washington, D. C.
Lockridge, R. W	Student—University of Maryland	College Park, Md.
Lord, E. C. E	U. S. Bureau of Public Roads	Washington, D. C.
Love, H. J	National Slag Association	Cleveland, Ohio.
Lysle, F. B	West Pa. Cement Company	Butler, Pa.
MacDonald, Thos. H	U. S. Bureau of Public Roads	Washington, D. C.
McClure, A. E	U. S. Bureau of Public Roads	Washington, D. C.
McComas, Wm. E	Portland Cement Association	Philadelphia, Pa.
McDermott, J. R	State Road Commission	Keyser, W. Va.
McGuire, D. D	State Highway Department	Nashville, Tenn.
McKay, James H	State Roads Commission	Baltimore, Md.
McKee, S. C	Truscon Steel Company	Toledo, Ohio.
Mack, W. W	State Highway Department	Dover, Del.
Marr, R. A., Jr	Virginia Military Institute	Lexington, Va
Marraffa, M. V	Warren Brothers Road Company	Washington, D. C.
Marston, Anson	Iowa State College	Ames, Iowa.
Martin, Geo. E	The Barrett Company	New York City.
Mattimore, H. S	State Highway Department	Harrisburg, Pa.
Mebus, Chas. F	Construction Engineer	Glenside, Pa.
Meyer, Carl F	Worcester Polytechnic Institute	Worcester, Mass.
Milburn, H. M	U. S. Bureau of Public Roads	Washington, D. C.
Miller, H. H	Department of Highways	Harrisburg, Pa.
Mills, W. H., Jr	State Highway Department	Columbia, S. C.
Morris, Mark	State Highway Commission	Ames, Iowa.
Morrison, R. L	University of Michigan	Ann Arbor, Mich.
Mullen, C. S	State Highway Department	Richmond, Va.
Mullis, Ira B	U. S. Bureau of Public Roads	Washington, D. C.
Myers, Bert	Iowa State Highway Commission	Ames, Iowa.
Myers, J. E	Department of Public Works	Albany, N. Y.
Nelson, R. E	West Virginia University	Morgantown, W. Va.
Nethercut, E. S	Western Society of Engineers	Chicago, Ill.
Ogden, Wm.	Highway Industries Association	Washington, D. C.
Older, Clifford	Consoer, Older & Quinlan	Chicago, Ill.
Olson, H. R	Chicago Regional Planning Ass'n	Chicago, Ill.
Oram, H. P	Assistant Engineer Commissioner	Washington, D. C.
Parsons, D. A	U. S. Bureau of Standards	Washington, D. C.
Pauls, J. T	U. S. Bureau of Public Roads	Washington, D. C.
Peabody, L. E	U. S. Bureau of Public Roads	Washington, D. C.
Perkins, Wm. C	National Paving Brick Mfgrs. Ass'n.	Langhorne, Pa

Perry, Lynn	Lafayette College	Easton, Pa.
Perry, T. R.	State Highway Commission	Ames, Iowa.
Pettingell, E. D.	Philadelphia Quartz Company	Philadelphia, Pa.
Phillips, E. W.	Engineer	Wilmington, Del.
Phipps, G. T.	Student—University of Maryland	College Park, Md.
Phoenix, J. W.	State Highway Commission	Raleigh, N. C.
Pitighiami, Fausts R.	D. Pitighiami	Roma, Italy.
Pollock, C. D.	Granite Paving Block Mfgrs. Ass'n.	New York City.
Preston, H. K.	University of Delaware	Newark, Del.
Price, M. M.	Student—University of Maryland	College Park, Md.
Priddy, B. E.	Chas. Havass & Company, Inc.	New York City.
Proudley, C. E.	National Sand & Gravel Association	Washington, D. C.
Rader, L. F.	Brooklyn Polytechnic Institute	Brooklyn, N. Y.
Rea, A. S.	State Highway Department	Columbus, Ohio.
Reagel, F. V.	State Highway Department	Jefferson City, Mo.
Reimer, F. A.	Consulting Engineer	East Orange, N. J.
Reshm, G. H.	Department of Highways	Harrisburg, Pa.
Rhine, E. B.	U. S. Bureau of Public Roads	Washington, D. C.
Richart, F. E.	University of Illinois	Urbana, Ill.
Rieman, E. H.	Grasselli Chemical Company	Cleveland, Ohio.
Roalf, Thos. H.	State Board of Public Roads	Providence, R. I.
Robertson, R. W.	Standard Lime and Stone Company	Baltimore, Md.
Roche, W. R.	Grasselli Chemical Company	Cleveland, Ohio.
Root, B. P.	U. S. Department of Commerce	Washington, D. C.
Sarason, S. D.	Syracuse University	Syracuse, N. Y.
Sarason, Mrs. S. D.		Syracuse, N. Y.
Schappler, R. C.	State Highway Department	Jefferson City, Mo.
Schlesinger, G. F.	National Paving Brick Mfgrs. Ass'n.	Washington, D. C.
Schmitt, F. E.	Engineering News-Record	New York City.
Schnepfe, F. E.	Consulting Engineer	Washington, D. C.
Scholer, C. H.	State Highway Commission	Manhattan, Kans.
Schumacher, L. B.	Jones & Laughlin Steel Corp.	Pittsburgh, Pa.
Scofield, H. H.	Cornell University	Ithaca, N. Y.
Scott, S. H.	American Tar Products Company	Pittsburgh, Pa.
Sehorn, H. F.	Student—University of Maryland	College Park, Md.
Seward, P. F.	American Road Builders' Association	Washington, D. C.
Shafer, Geo. E.	Armco Culvert Mfgrs. Association	Middletown, Ohio.
Sharp, H. M.	The France Stone Company	Toledo, Ohio.
Sherman, H. L.	Skinner, Sherman & Esselen, Inc.	Boston, Mass.
Shirley, H. G.	State Highway Department	Richmond, Va.
Slack, Searcy B.	State Highway Department	East Point, Ga.
Slater, W. A.	Lehigh University	Bethlehem, Pa.
Smith, Baxter	U. S. Bureau of Public Roads	Washington, D. C.
Smith, Earl B.	Iowa State College	Ames, Iowa.
Sourwine, J. A.	U. S. Bureau of Public Roads	Washington, D. C.
Spraragen, Wm.	National Research Council	New York City.
Stanton, C. B.	Carnegie Institute of Technology	Pittsburgh, Pa.
Starr, R. C.	National Sand & Gravel Association	Washington, D. C.
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Steinberg, S. S.	University of Maryland	College Park, Md.
Steljes, Martin	Concrete Steel Company	New York City.

Stephanian, Stephen	The Arrow Sand & Gravel Company	Columbus, Ohio.
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Stephenson, S. A., Jr.	Rutgers University	New Brunswick, N. J.
Stericker, Wm.	Philadelphia Quartz Company	Philadelphia, Pa.
Stewart, Hamilton	Contractor	Wilmington, Del.
Stone, J. H.	U. S. Bureau of Public Roads	Washington, D. C.
Strahan, C. M.	University of Georgia	Athens, Ga.

Talbot, K. H.	Koehring Company	Milwaukee, Wis.
Tarwater, E. L.	U. S. Bureau of Public Roads	Washington, D. C.
Taylor, A. R.	American Tar Products Company	Pittsburgh, Pa.
Taylor, T. Warren	Union College	Schenectady, N. Y.
Teller, L. W.	U. S. Bureau of Public Roads	Washington, D. C.
Terrell, D. V.	University of Kentucky	Lexington, Ky.
Thomas, S. C.	U. S. Bureau of Public Roads	Washington, D. C.
Thompson, J. Trueman	Johns Hopkins University	Baltimore, Md.
Thoreen, R. C.	U. S. Bureau of Public Roads	Columbus, Ohio.
Thornton, H. I.	American Tar Company	Malden, Mass.
Tilden, C. J.	Yale University	New Haven, Conn.
Tiney, B. C.	State Highway Department	Lansing, Mich.
Toner, A. C.	Portland Cement Association	Washington, D. C.
Torkelson, M. W.	State Highway Commission	Madison, Wis.
Trainor, L. S.	McEverlast, Inc.	Chicago, Ill.
Troxell, G. E.	University of California	Berkeley, Calif.
Tucker, Harry	North Carolina State College	Raleigh, N. C.
Tucker, John, Jr.	U. S. Bureau of Standards	Washington, D. C.

Ulman, M. H.	Department of Highways	Harrisburg, Pa.
Upham, Chas. M.	American Road Builders' Association	Washington, D. C.

Vassiliev, S. A.	Russian Ass'n of Engineers	New York City.

Waldrop, J. D.	State Highway Commission	Raleigh, N. C.
Walker, G. W.	U. S. Bureau of Standards	Washington, D. C.
Walker, Stanton	National Sand & Gravel Association	Washington, D. C.
Wallace, J. N.	Student—University of Maryland	College Park, Md.
Weathers, H. C.	State Road Department	Gainesville, Fla.
Wheeler, H. E.	U. S. Bureau of Public Roads	Washington, D. C.
Whitcraft, L. N.	Portland Cement Association	New York City.
Whitehurst, H. C.	District Building	Washington, D. C.
Wiggin, A. J.	State Highway Commission	Augusta, Me.
Wilder, E. A.	U. S. Bureau of Public Roads	Washington, D. C.
Wildman, J. P.	U. S. Bureau of Public Roads	Washington, D. C.
Williams, C. E.	Manufacturers Record	Baltimore, Md.
Willis, E. A.	U. S. Bureau of Public Roads	Washington, D. C.
Wilson, W. S.	Student—University of Maryland	College Park, Md.
Wise, W. F.	Southwest Stone Company	Dallas, Texas.
Withey, M. O.	University of Wisconsin	Madison, Wis.
Woolf, D. O.	U. S. Bureau of Public Roads	Washington, D. C.
Woolley, C. L.	State Board of Roads	Providence, R. I.
Worcester, Wm. E.	Kinney Manufacturing Company	Philadelphia, Pa.
Worley, J. S.		Ann Arbor, Mich.
Worthen, A. L.	The Connecticut Quarries Company	New Haven, Conn.
Wright, C. A.	U. S. Bureau of Standards	Washington, D. C.

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